Recent studies and other research have indicated the likelihood and magnitude of a potentially large seismic event along the Wasatch Front. As this information has come to light, innovations in the means and methods of seismic-resistant design and construction have also been developed. Unfortunately, many of the structures designed and built prior to these innovations, such as the Utah State Capitol Building, do not have the capability to remain intact and safe if subject to a significant seismic event. Although the original design of the Capitol Building was very innovative, it occurred at a time when seismic-resistant design and knowledge of seismic potential were virtually nonexistent. As a result, the Utah State Capitol does not have the characteristics that would enable it to perform adequately during a significant earthquake.

1. Purpose and Scope

The purpose of this report is to provide a summary of the seismic evaluation and to propose methods of improving life safety of the Utah State Capitol Building against anticipated seismic activity. In addition, because of its historic nature, there is considerable interest in minimizing damage to the building and its contents.

A study with similar objectives was performed in 1993 by Reaveley Engineers and Associates[1] but was limited in scope and extent of analysis. This report consists of an evaluation and extension of the analysis and recommendations proposed in the 1993 report. The intent herewith is to carry the analysis performed previously to the next level by creating sophisticated computer models of the as-is structure and to explore the possible methods of strengthening or altering the structure to minimize the potential for loss of life and damage due to seismic activity. This study incorporates the latest advancements in computer aided analysis in conjunction with current building codes and standards along with site specific geoseismic engineering.

a. Seismic Potential and Structural Vulnerability

In recent years many geoseismic discoveries have been made along the Wasatch Front. Perhaps the most significant is that of frequent seismic activity along the east benches of Salt Lake and Utah Valleys. Recent advancements in seismic instrumentation and equipment sensitivity have revealed that there are approximately 700 measurable earthquakes in the state of Utah each year. Most of these earthquakes go unnoticed due to their low magnitude and relative intensity. However, large earthquakes do occur in Utah at relatively long recurrence intervals. Studies indicate that a large earthquake of Richter Magnitude 6.5 to 7.5 occurs along the Wasatch Fault about once every 350 years. An earthquake of this magnitude is expected to occur on the Salt Lake segment of the Wasatch Fault about once every 1300 years.

Many structures near the Wasatch Fault were constructed at a time when there was little or no knowledge of the potential seismic activity. In addition, the standards of structural design and construction in terms of seismic safety have only recently become a part of standard building practice in terms of typical building life spans. For these reasons, many of the structures near the Wasatch Fault, including the Utah State Capitol are particularly vulnerable to life threatening damage due to seismic activity.

1. Reaveley Engineers & Associates, *UTAH STATE CAPITOL BUILDING SEISMIC RETROFIT STUDY*, Salt Lake City, UT (1993).

b. Summary of 1993 Report

As part of the current evaluation, the report prepared in 1993 is under review. The purpose of the investigation and study performed in the 1993 evaluation was to:

- Evaluate the seismic strength of the structure.
- Report unsafe conditions.
- Provide recommendations for future planning and possible methods of seismically upgrading the structure.

Also included in the scope of the structural evaluation were the following:

- Obtain and study existing plans where available.
- Perform on-site observations to compare plans to actual conditions and observe areas for which plans were not available. Not all existing conditions could be verified due to lack of accessibility or the need for demolition to expose structural members.
- List major seismic deficiencies and assess vulnerability.
- Provide recommendations and options for upgrading the structure.
- Provide a preliminary cost estimate for the seismic upgrade options.

c. Limitations of 1993 Report

Although worthwhile, the 1993 study was subject to limitations primarily due to the available building codes and means and methods of analysis for the structure. The analysis, recommendations and other information provided in the 1993 report are subject to the following limitations:

1. At the time of the 1993 study there was not a nationally accepted standard for the evaluation and analysis of existing buildings consisting of reinforced concrete structural frames. The authors of the previous report recognized this limitation and stated:

   "Presently the only 'code' for retrofit construction is the Uniform Code for Building Conservation (UCBC) which applies mainly to unreinforced masonry bearing wall buildings. The Capitol Building does not fall under the UCBC because it is a reinforced concrete frame building with unreinforced masonry infill walls. No retrofit code has been completed for this type of building. Retrofit codes are currently in development and will be available in the next few years. For this reason there is no formally adopted standard of retrofit for the Capitol Building….. Where feasible, the standards of the Uniform Building Code (UBC) which applies to new construction, have been used to analyze possible retrofit options and calculate earthquake forces."

   For the 1993 study, lateral analysis was performed using the 1991 edition of the Uniform Building Code[2] which contains standards prescribed for new construction. Retrofit codes such as NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS[3] (FEMA 273) have recently been released for use by engineers and owners for the evaluation of existing structures. These codes have been written specifically for the

2. International Conference of Building Officials, *Uniform Building Code UBC-91*, Whittier, CA.

3. FEMA, 2000, NEHRP *Guidelines for the Seismic Rehabilitation of Buildings*, Publication No. ASCE/FEMA 273 Prestandard SECOND SC DRAFT, Federal Emergency Management Agency, Washington, D.C.

evaluation of existing structures. Since structures such as the Capitol Building cannot meet the prescriptive demands of new building codes, documents such as FEMA 273 have been developed. Codes such as this enable the evaluation of the structure based on the required level of seismic performance (level of acceptable post-earthquake damage) that is defined by the owner and/or building official. FEMA 273 was used as the base document for the current evaluation and analysis of the Utah State Capitol Building.

2. Due to insufficient software development and computational power, the 1993 study did not include an extensive three-dimensional structural analysis of the building. Although the methods of analysis used for the 1993 study were sound and produced worthwhile results, the available computational power was not enough to mathematically explore the analysis and retrofit possibilities to the degree now possible.

3. The 1993 study included only a preliminary geotechnical study. This preliminary study did not include a field exploration of the building site and was primarily written to provide a general indication of the site's soil characteristics. Extensive testing was deemed beyond the scope of the 1993 study. Geotechnical analysis methods used in 1993 tended to follow a more code-driven prescriptive path than a site specific path. Rather than using statistical tools to determine the level of probable ground motions for a specific site, earthquake forces were based simply on seismic zones and localized soil conditions.

4. The 1993 study included only a limited study of existing material strengths and properties. Of the concrete specimens taken, all were from the dome. For the current study, samples of concrete, concrete reinforcing steel, and structural steel have been extracted from various parts of the structure including the dome. These samples have been analyzed and the results were used as part of the material properties input into the computer model.[4] Knowledge of the structure's material properties leads to a more accurate analysis of the building's overall seismic behavior.

Even though these limitations prevented a more precise analysis of the structure for the 1993 study, the conclusions reached are of value and are not unlike the results and recommendations of the current study. One of the primary purposes of the current study is to review, evaluate, and build upon the results and recommendations from the 1993 report.

4. REPORT, *Materials Testing Services Existing Material Conditions*, AGRA Earth and Environmental, Salt Lake City, UT, June 2000.

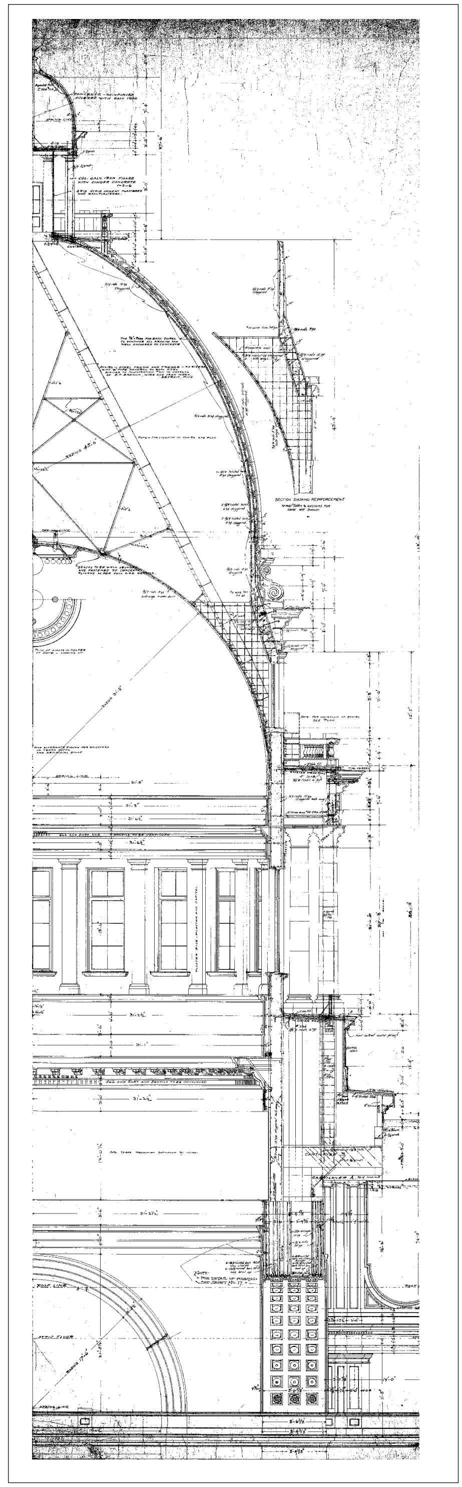
2. Innovations in Methods and Codes Since 1993

Innovations in building codes as well as the tools available for performing analyses have significantly improved since the 1993 study. Because of the advent of nationally recognized retrofit building codes in addition to the increased power of desktop computers and software, engineers are able to evaluate structures and determine their characteristics in far greater detail than was previously possible. These innovations have been incorporated into the existing study and include:

- The 3-dimensional computer modeling and evaluation of the building structure as a whole, including the evaluation of the existing structure as well as the evaluation of each of the possible retrofit schemes herein discussed. Due to the complex nature of this structure, it was not previously possible to perform analyses such as this on relatively small desktop computers without significantly simplifying the model, thus compromising the accuracy of the evaluation. Since computational power has increased so dramatically since 1993 it is now possible to explore multiple evaluations and retrofit schemes in relatively little time and with greater accuracy. For the current study, version 7 of SAP2000 was used to perform the computer modeling.
- Site specific earthquake analysis using specific soil characteristics, fault proximity, and ground motions based on geotechnical explorations, statistical data, and previously recorded earthquakes. USGS (United States Geological Survey) has recently released earthquake acceleration contour maps that provide the magnitude of ground motions for a specific location that are probable for a certain statistical time interval. Whereas previous techniques used seismic 'zoning' to assess the level of earthquake load to use for analysis, the contour maps enable a much more precise site specific determination of seismic ground motions that could occur for the building. As part of the current study, extensive geotechnical and geo-seismic evaluations have been performed.[5] These studies have provided considerably more information than previous studies and include (among other information); site specific soil classification, soil shear wave velocities, soil bearing capacities, site specific response spectra and site specific time history accelerograms developed from previously recorded earthquakes. This information enables a much more in-depth analysis of the specific characteristics of the building site and how these characteristics will influence overall structural dynamics.
- Evaluation of the as-is capacity of the existing structure based on new analysis methods of FEMA 273. Rather than basing the capacity of the structure and its components on analysis methods and seismic loads used for the design of new structures, FEMA 273 includes nationally recognized analysis methods and seismic loading that has been specifically developed for the evaluation of existing structures. Often referred to as 'pushover' analysis, this new method is an evaluation of the actual behavior of a structure as it relates to a specific level of seismic motion. For this method of analysis, the computer generated model is actually 'pushed over' to the point that it begins to collapse. This enables the evaluation of existing building capacities which are far more accurate and descriptive than were previously possible.

5. REPORT, *Site-Specific Time Response Spectra Existing Utah State Capitol Building*, AGRA Earth and Environmental, Salt Lake City, UT, July 2000.

The innovations outlined above have enabled evaluations of the structure and possible retrofit schemes that are far more accurate and descriptive than were possible in 1993. The results of the analyses using these innovations are herein documented and are used as the primary basis of the recommendations for the life safety performance and seismic protection of this important facility.



1. STRUCTURAL SYSTEMS

The standards, criteria and objectives for the restored building are:

- Life Safety
- Function - Efficiency / Effectiveness
- Historic / Architectural Integrity

a. Life Safety

1) STANDARD: The restored building must meet the minimum recommendations of NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS (FEMA 273).

a) Objective: Upgraded structure must perform to a "Life Safety" level of seismic performance for a seismic event having a 10% probability of being exceeded in a 50 year period.

b) Objective: Upgraded structure must perform to a "Collapse Prevention" level of seismic performance for either a seismic event having a 2% in 50 year probability of being exceeded or for the characteristic large, rare earthquake for the Wasatch Front (Maximum Considered Earthquake).

b. Function - Efficiency / Effectiveness

1) STANDARD: The building must meet Life Safety rehabilitation objectives without affecting the standard function and purpose of the facility

a) Objective: The retrofit measures should not be intrusive to functional spaces and should not affect the ability of the occupants to effectively perform their duties.

c. Historic / Architectural Integrity

1) STANDARD: The historic integrity of the Capitol must be preserved.

a) Objective: Pursue retrofit measures that have minimal impact on historic fabric that can also reduce the potential for seismic damage to the historic fabric.

2. Building Codes

Many structures throughout the country are similar in nature to the Utah State Capitol, and like the Capitol cannot be deemed adequate by the prescriptive standards of modern codes. Because of this, the Federal Emergency Management Agency (FEMA) as part of NEHRP (National Earthquake Hazards Reduction Program) has developed performance standards for the analysis and seismic rehabilitation of existing buildings. The latest of these standards is the document FEMA 273 entitled "NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS." This document specifies nationally recognized provisions for the seismic rehabilitation of buildings. It is intended to be suitable for use by qualified professional engineers as a guideline reference to be used with careful consideration and engineering judgement.

Unlike traditional building codes which use prescriptive means and methods of construction and rehabilitation, FEMA 273 uses a methodology based on a selected performance objective. Prescriptive means of building construction and rehabilitation typically dictate the minimum design and constructions standards that are to be met, sometimes with little or no regard to the specific circumstances of the project or the building performance goals desired by the owner. Performance based methods base the level of design and retrofit on the specific performance goals established by the owner which are then approved by the building official. The levels of structural performance that an owner can choose vary from Operational (O) at one extreme to Collapse Prevention (CP) at the other. As part of the performance based methodology, the owner of a building is also required to select the level of seismic force or seismic hazard to use in the evaluation of the facility. The level of seismic load is typically based on an a characteristic earthquake that has a certain statistical probability of being exceeded for a given period of time.

a. Levels of Structural Performance

Among the various structural performance levels defined by FEMA 273 are several discrete levels called Operational (O), Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP).

For the Operational level of performance (O) there is very little (if any) damage after a significant seismic event. The overall structure remains serviceable and all backup utility services maintain their functions. This level of performance is rarely used since it requires significant costs and extreme measures to make a structure perform to this level. This standard is usually too far above the owner's goals to be included as part of the performance objective.

The Immediate Occupancy (IO) level of performance is defined as a structure having very light overall damage after a seismic event of certain magnitude. The structure experiences no permanent deformations and retains is original strength and stiffness. It may experience minor cracking of facades, partitions, ceilings and structural elements with little or no damage to elevators and fire protection systems. The equipment and contents are generally secure but may not operate due to mechanical failure or lack of utilities.

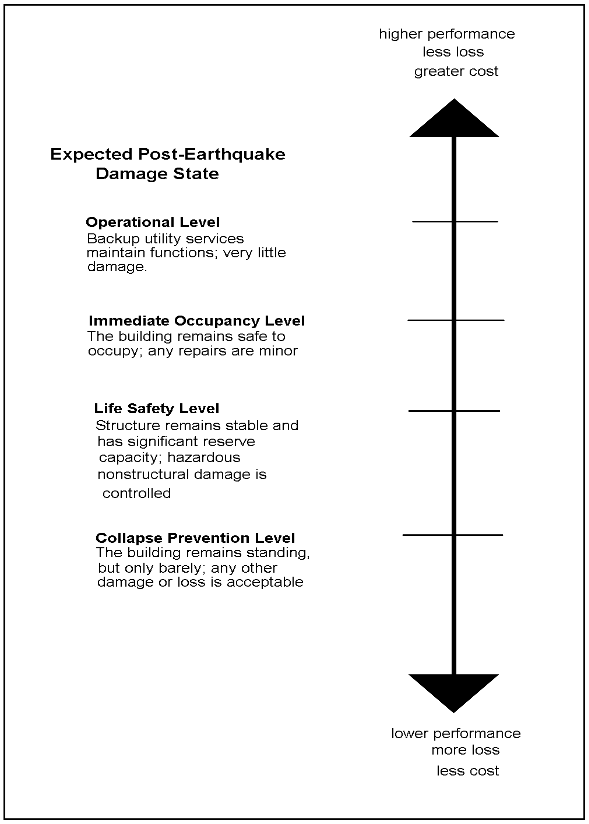


Figure 1 Comparative Summary of Performance Levels

For the Life Safety (LS) level of performance, overall damage is moderate. Residual strength and stiffness is maintained on all stories with functioning gravity load bearing elements. For this level of performance the parapets remain intact and do not become falling hazards. The structure may experience some permanent deformation with considerable damage to partitions. Although the overall structure may remain stable and the occupants may survive, the building may be beyond a state of feasible repair after performing to a Life Safety (LS) level of performance. Falling hazards for this level of performance may be mitigated but many architectural, mechanical and electrical systems may be damaged.

For the Collapse Prevention (CP) level of performance the objective is for the structure to remain standing after a seismic event of certain magnitude. For this level of performance there is little residual stiffness and strength. Load bearing walls and columns function and are still intact. There are large permanent deformations, some exits are blocked and infills and parapets failed or are near failure. The overall building is near collapse and is beyond a state of feasible repair, with extensive damage to all structural and nonstructural components.

Figure 1 provides a comparative summary of the basic performance levels defined by FEMA 273. The costs associated with each level of performance are directly reflected by the level of performance desired. As the performance level increases, so do the relative costs of retrofit. However, with increased seismic performance comes less cost associated with post-earthquake repair and restoration.

b. Seismic Hazard

When using the performance based methodology of FEMA 273, the owner of a building is required not only to select a level of desired performance, but to also select the magnitude of seismic hazard for which the specific performance level is desired. Seismic shaking hazards may be formed considering ground motions with any defined probability of exceedance, or based on any deterministic event on a specific fault. A probabilistic event is defined by using statistical analysis methods in conjunction with geoseismic trenching investigations and geotechnical analysis. A deterministic earthquake hazard is based on a characteristic seismic event that occurs over an average time interval for a specific fault. Probabilistic events are defined on a "likelihood of exceedance" basis. For example, a design standard probabilistic earthquake could be defined as an event have a 10% probability of being exceeded in a 50 year period. In basic terms this means that 90% of the earthquakes occurring over a 50 year period are likely to have ground motions that are less severe than that corresponding to the design standard. An example of a deterministic event would be the characteristic large earthquake of Richter magnitude 6.5 to 7.5 that occurs along the Wasatch Fault about once every 350 years.

FEMA 273 recommends two basic levels of seismic shaking hazard defined as Basic Safety Earthquake 1 (BSE-1) and a more severe Basic Safety Earthquake 2 (BSE-2). BSE-1 is also referred to as the Design Basis Earthquake (DBE) by many building codes and is defined as horizontal ground shaking corresponding to an earthquake having a 10% probability of being exceeded in a 50 year period. This earthquake has a return period 474 years which means this earthquake has a probability of occurring once during a period of 474 years.

For the BSE-2 the level of ground shaking can be defined by either the level of horizontal motion of an event having a 2% probability of being exceeded in a 50 year period or the horizontal motion of the deterministic Maximum Considered Earthquake (MCE) which is roughly the equivalent of the 2% in 50 year event. The 2% in 50 year seismic event has a return period of 2475 years. The MCE is classified as the characteristic large, rare earthquake. For the Wasatch Front, the MCE occurs approximately once every 350 years. For specific segments of the Wasatch Fault such as the Capitol Hill Fault Zone, the MCE occurs about once every 1300 years.

For the current evaluation and analysis of the Utah State Capitol, the performance based methods of FEMA 273 were implemented. FEMA 273 recommends that a minimum level of required performance called the "Basic Safety Objective (BSO)" be established for analysis and rehabilitation. A Basic Safety Objective consists of selecting a post earthquake damage state (performance level) then selecting a seismic hazard for which the specific level of performance is desired. FEMA 273 guidelines recommend that the minimum Basic Safety Objective for the Capitol Building be a Life Safety (LS) level of performance for the BSE-1 (10% in 50 year seismic event) and a Collapse Prevention (CP) level of performance corresponding to the BSE-2 (MCE or 2% in 50 year seismic event). The most cost effective methods for the Basic Safety Objective may also have the added benefit of protecting the structural and nonstructural components of the building from damage and help preserve the historic fabric of the structure. In terms of FEMA 273 this would be considered an Enhanced (Life Safety) Rehabilitation Objective because it provides a level of performance that is above and beyond the minimum requirements that would constitute a Basic Safety level of performance.

1. SITE CHARACTERISTICS AND SEISMICITY

The Utah State Capitol is located in the intermountain zone very near the Warm Springs trace of the Capitol Hill Fault Zone. Geotechnical analysis of site conditions (and USGS maps) indicate that liquefaction is not likely at the site. For the Maximum Considered Earthquake (MCE) level of seismic hazard, the USGS contour maps for seismic ground motion indicate that the Capitol Building could experience lateral seismic ground accelerations in excess of 73% of the force due to gravity. This means that the horizontal loads on the structure could be as high or higher than 73% of its own weight for this level of seismic ground motion. Figure 2 shows the contour map for the MCE earthquake and the location of the Utah State Capitol in relation to the seismic contours. By way of comparison this level of lateral acceleration is not unlike that expected by many areas in California.

The site investigation also includes the development of site specific seismic characteristics based on the geotechnical investigation (see Appendix). Recent advancements in the field of geoseismic engineering have enabled the production of theoretical seismic ground motions based on the unique characteristics of the site in conjunction with other seismic data provided by USGS. Other geotechnical information includes the site classification. The geotechnical investigation indicates that the site is best classified with a type D soil. Soil type D is defined by FEMA 273 as a stiff soil that transmits shear waves at a rate between 600 and 1200 feet per second. This soil type generally has stable properties and can behave in an elastic manner under normal service load.

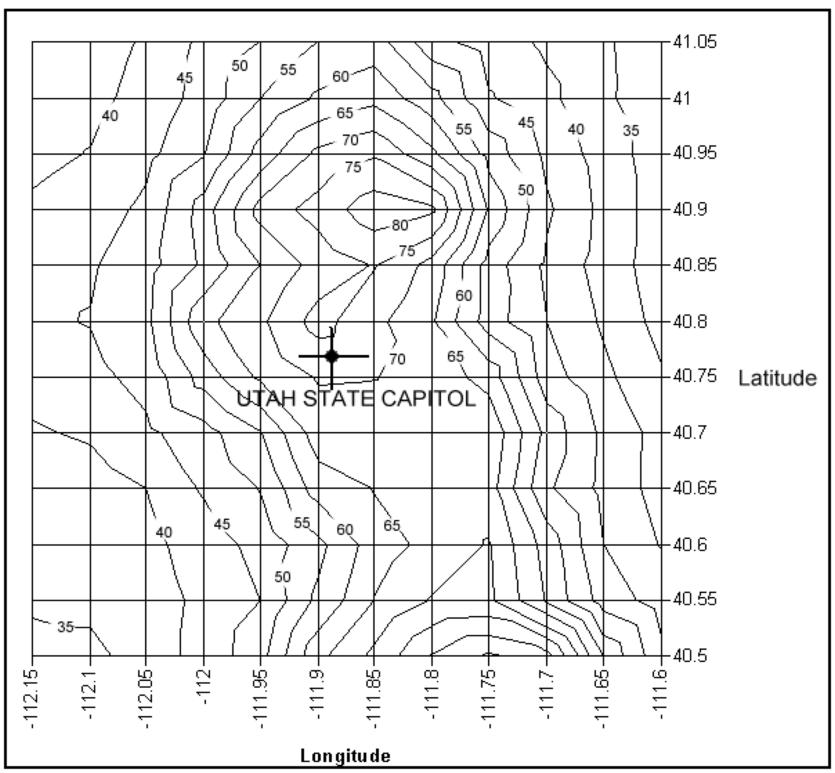


Figure 2 Spectral Acceleration Contours for MCE and 1 Second Period

2. BUILDING CHARACTERISTICS

a. Basic Structure

The foundation of the structure consists of continuous perimeter foundation walls resting on spot footings spaced typically at 14’ on center with plynths cast atop the concrete footings which support concrete columns above. The footings at the west end of the structure are at a lower elevation than the footings at the east end of the structure. This results in a greater ceiling height in the basement at the west end of the structure than at the east end.

Two different basic floor systems are used in the structure. The ground floor and the roof consist of five-inch-thick reinforced concrete slabs supported by concrete girders that span between and connect to the concrete columns. At the intermediate floors and at the attic, the floor system consists of concrete girders that span from column to column and a series of concrete T-beam/purlin system that spans one-way between each of the girders. The concrete slab or flange portion of the T beam is 2-1/2” thick. The concrete for this system was formed using a network of hollow clay tiles to fill the void spaces between the concrete T beams. Unlike modern construction where forms are removed and reused, the clay tiles forming the void spaces were left in place and are now part of the permanent structure, though they are of little benefit structurally.

The large skylights provide an abundance of natural light for the main atrium and rotunda and also the senate, and legislative chambers. Above the skylights in the main atrium of the structure and above the legislative and senate chambers a series of steel trusses are used to support the skylights and suspended ceilings. The trusses span the large floor and attic openings at the roof level of the structure and are typically composed of double angle web members with some curved bottom chords and straight top chords.

The dome at the center of the structure is supported by four hollow, triangular shaped, reinforced concrete piers consisting of two 4’ x 8’ concrete columns and 12” thick concrete walls (see Figure 3 for plan of structure supporting dome). The combination of walls that have been cast monolithically with the columns results in this portion of the structure being very stiff and rigid compared to the remainder of the structure. The columns and walls are connected at the top on all four

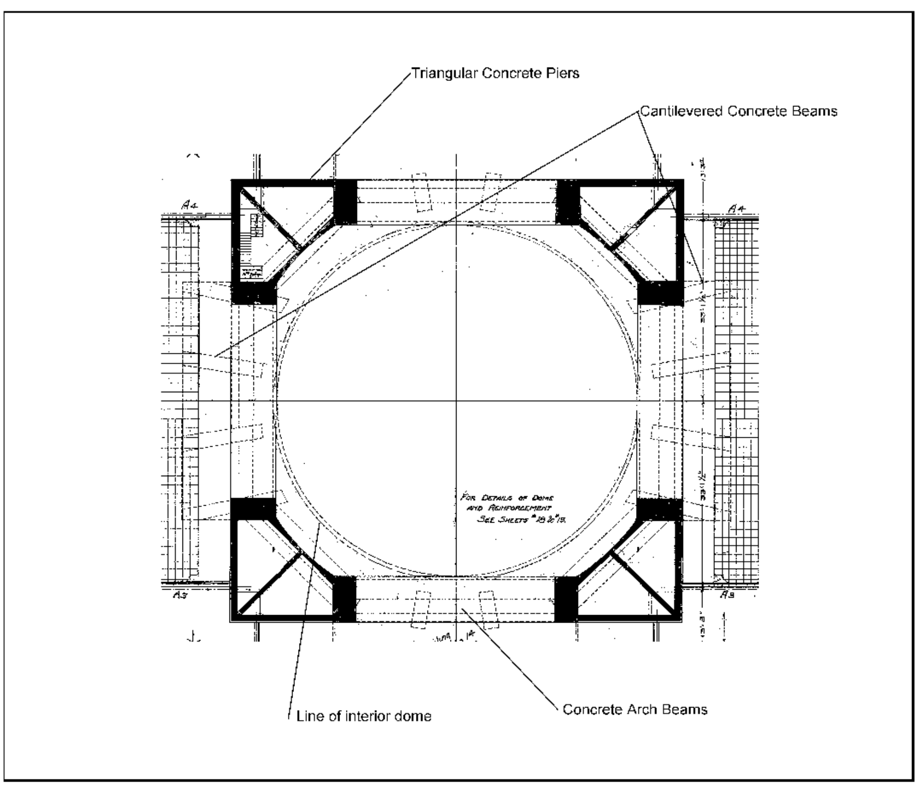


Figure 3 Plan of Structure Supporting Dome

sides by a large arch beam that is approximately 8 feet wide and 10 feet deep. Figure 4 shows an elevated section view of the dome structure taken from the original drawings. Though the final design varied slightly from the configuration shown in Figures 3 and 4, the overall scheme, look, and primary structure of the dome is essentially the same. The reason for adding the steel trusses to the upper portion of the dome is not completely understood, however it is believed that the trusses were added to support the concrete construction of the dome in addition to providing a support mechanism for the inner dome structure and other nonstructural components.

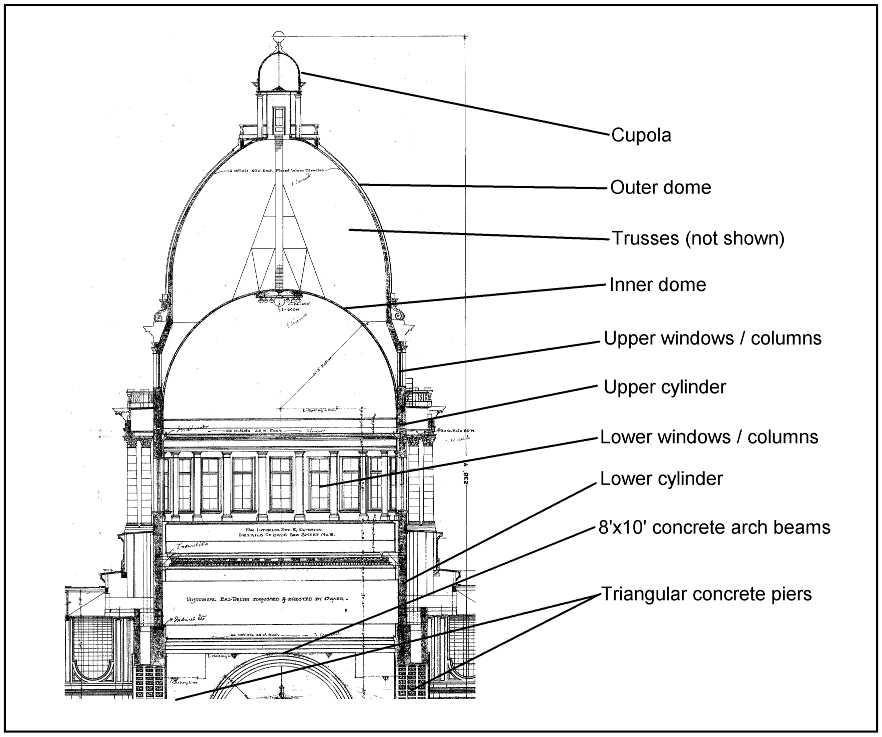
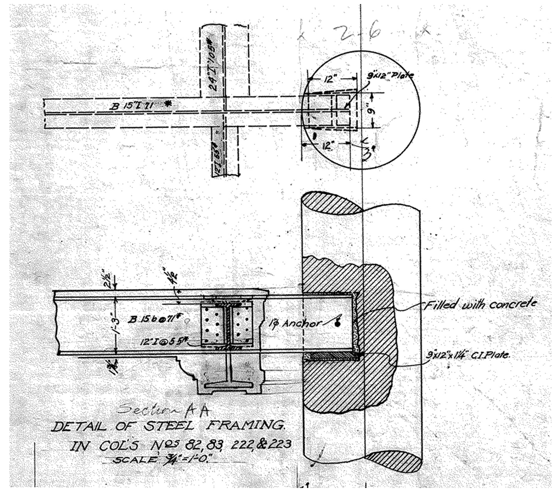


Figure 4 Elevated Section View of Dome Structure

Although the reinforced concrete frame provides the primary support network for the structure, there are some locations where concrete walls are used as bearing walls between floors. This occurs primarily at the lower levels and vault locations. For the original building the interior partitions consist of hollow clay tile with plaster facing. The original drawings in conjunction with field observations indicate that hollow clay tiles and unreinforced brick masonry have been used at many locations to act as partitions, backing material for plaster and other architectural finishes, or simply as infill. Though walls such as these can provide considerable initial stiffness, they tend to crumble under very small seismic movement and therefore do not benefit the structure seismically because they do not remain intact beyond the initial seismic impact.

On the south, east, and west exteriors of the building granite columns have been used which extend from the first floor to the attic level of the structure. Though primarily an architectural feature, the columns are used to support the loads of the attic and roof levels. The granite columns have been assembled from cylindrical sections approximately 3'-9" in diameter that are stacked and doweled together with a single smooth dowel 1" in diameter.

At the interior of the structure around the perimeter of the rotunda and the east and west halls are similar columns with a similar function and slightly different composition. These columns are seamless marble approximately 2'-7" in diameter and support loads from the second and third floor balconies. The structure of the second floor is grouted into a pocket of the marble columns and anchored with a 1" diameter dowel. At the third floor balcony the structure rests atop the marble columns. The anchorage of the third floor to the top of the marble columns is not clear from the original drawings nor is it easily discernible from field observations. Based on other detailed and observed conditions, it is likely that the third floor structure simply rests atop the marble columns with little, if any positive anchorage.



ONE EARLY DRAWING OF THE COLUMN BRACING

b. Nonstructural Components

The nonstructural elements and components of this facility are considered very heavy by modern standards and contribute significantly to the overall mass that affects the building's response to seismic motion. Since mass is one of the primary components that determine the natural motion of a structure, the unusually heavy structural massing of this building leads to very large overall seismic loads.

1. Exterior

The exterior cladding of the structure consists of very heavy, shaped granite that was stacked and anchored to brick and hollow clay tile masonry backing. The existing condition of the anchors is unknown, but is likely poor and inadequate seismically. The masonry backing is unreinforced and thus provides little benefit seismically for the structure and its cladding.

The parapets at the roof level are unusually tall and are not braced. Like the typical exterior cladding on the remainder of the building, they consist of sections of shaped granite stacked and pinned together in combination with unreinforced brick masonry to form the parapet assembly. Many of the other exterior walls and pediments at the roof level are composed of either unreinforced brick masonry or clay tile that is unbraced.

The dome of the capitol building is clad primarily with shaped granite and terra cotta. The terra cotta bands were used primarily for ornate purposes. The cupola atop the dome consists of an unbraced balustrade and is covered with copper.

2. Interior

The grand east and west hallways as well as the central rotunda itself is covered with very heavy marble elements. At the ends of the east and west hallways are concrete stairways connecting from level to level which are also clad with marble. Though the original design and construction of the interior cladding assembly may have been very innovative at the time of construction, it is likely insufficient in terms of modern methods and practices.

The original partitions for the building were composed of either unreinforced masonry or hollow clay tile infills. Many of these walls are still in place and perform well under normal everyday service loads. However, because of their brittle nature, these walls are not likely to perform well seismically because they could crumble under low seismic load.

The original ceilings of the structure are composed of suspended metal lath and plaster. This ceiling system is considerably heavy by modern standards and could become a significant threat to life should it become detached from the structure due to earthquake motion and become a falling hazard.

3. METHOD OF ANALYSIS

a. Materials Testing

Testing of the structural materials used in construction of the Utah State Capitol indicate standard materials with typical strengths and structural characteristics for a structure of this age and nature. Concrete cylinder testing and Windsor Probe testing indicate that concrete compressive strengths for the structure (not including the dome) range from about 2300 to about 3400 psi with one test at approximately 5200 psi. Cylinder tests for concrete extracted from the dome are considerably less and are as low as 900 psi. Though it is not known why the concrete strength of the dome is so low, it is most likely due to the placement of concrete at such a precarious location, or because of other environmental conditions at the time of construction. Table 1 lists the results of the Windsor Probe and laboratory testing for concrete specimens taken from various portions of the structure.

Test No.	Test Location	Windsor Probe Compressive Strength (psi)	Laboratory Compressive Strength (psi)
1	1st Floor - SW Corner	3,250	5,262
2	2nd Floor - NE Corner	2,476	2,271
3	3rd Floor - NE Corner	2,375	1,856
4	3rd Floor - NW Corner	2,800	2,493
5	4th Floor - NE Corner	2,800	3,046
6	4th Floor - SW Corner	2,800	3,490
7	Beam - North	2,475	3,140
8	Beam - South	2,475	2,720
9	Dome North No. 1	1,500	900
10	Dome North No. 2	1,400	1,470
11	Dome West No.1	--	1,800
12	Dome West No.2	1,725	2,720
13	Dome West No.3	2,050	1,030

Table 1 Summary of Concrete Specimen Compressive Strengths

b. Analysis Procedures

Determining the structure's seismic behavior is important in order to assess the nature and extent of retrofitting and/or altering that will be most effective in saving lives and limiting damage to this structure. Recent advancements in computer software and hardware have made it possible to model very complex structures and obtain results that are consistent with actual structural behavior. For the Utah State Capitol the seismic modeling was performed on SAP2000 Nonlinear Version 7. This application has the ability to model very large and complex structures and has enabled the creation of a three-dimensional virtual model of the State Capitol (see Figure 5). In addition, structural irregularities such as the dome of the State Capitol can be modeled and analyzed on state-of-the-art computer applications such as this. The configuration, size, and structural members of this model have all been input in accordance with geometry and other information shown in the original design drawings and the "as built" verified documents, in conjunction with observations of the building.

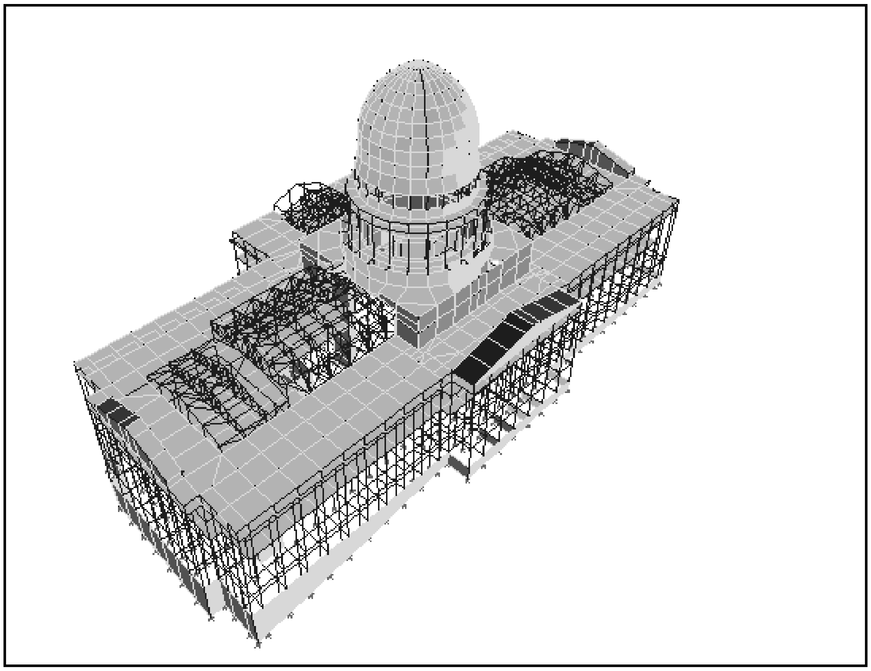


Figure 5 SAP2000 Structural Model of Utah State Capitol

SAP2000 allows for several different types of seismic analyses. Horizontal seismic loads may be input manually as static point loads at various locations on the structure or as dynamic response spectra loads or by recorded time history logs from actual earthquakes.

Static load analysis is the most simple and consists of placing horizontal static (constant) load on the structure and evaluating the effects. Though this method of analysis is the easiest way to perform structural seismic analysis it is often not the most appropriate because it does not capture dynamic load effects that can be caused due to the dynamic nature of an earthquake. Response spectrum analysis uses the various natural modes of vibration of a structure and combines the amount of seismic response for each of those modes based on a set of previously recorded or artificially generated earthquake ground motions. Time history analysis uses the lateral ground acceleration values recorded from a previous earthquake or values derived from artificially generated ground motion histories and applies them directly to the virtual model. For the State Capitol both response spectrum and time history analyses were used to analyze the structure and explore various retrofitting options.

The site specific geotechnical analysis provides the magnitude and nature of ground motions that are likely for the 10% in 50 year seismic event in addition to the Maximum Considered Earthquake (MCE). To assess the level and nature of horizontal ground motion associated with these seismic hazards, specific soil parameters determined from geotechnical boarings at the site are used. When used in conjunction with the probabilistic or deterministic ground motion data provided by USGS, it then becomes possible to determine the magnitude and nature of site specific earthquake ground motions that likely. In addition, ground motions recorded from previous earthquakes in other regions can be modified to reflect the likely ground motions at this site. The modified ground motions are then used to create artificial response spectra and time history records that are consistent with the expected seismic behavior at the Capitol Building. These records can then be applied to the computer model to assess the building's response to the anticipated ground motion. Applying the principles of FEMA 273 then enables the evaluation of the performance of the building based on its strength and stability at the level of earthquake motion correlating to the characterized seismic hazard.

4. ENHANCED REHABILITATION OBJECTIVE

Under special circumstances such as the evaluation of critical or historically significant facilities, an Enhanced Rehabilitation Objective is chosen by the owner. An Enhanced Rehabilitation Objective is selected to provide building performance superior to that of the FEMA 273 Basic Safety Objective (BSO) including damage control to nonstructural elements and contents as well as safety of the occupants. Since the FEMA 273 Basic Safety Objective is primarily concerned with saving the lives of the occupants of a structure, it does little to address the preservation of the structure itself. Under the BSO, the occupants of a structure may safely survive a significant earthquake, but the structure might be a total loss. Enhanced Rehabilitation Objectives are chosen to provide added protection for facilities that are themselves irreplaceable with contents or components that are also of significant value. Enhanced Rehabilitation Objectives usually use a higher level of seismic load and/or a more stringent acceptance criteria in terms of seismic damage. In some circumstances, an Enhanced Rehabilitation Objective can be achieved at a relatively low added cost to the Basic Safety level of upgrade. For the current analysis, recommendations for upgrade are primarily intended to meet the Life Safety objective or the Basic Safety Objective per FEMA 273. However, the methods of altering this structure herein recommended can provide an enhanced level of performance which will better protect the historic fabric at a minimal (if any) added cost to the "Life Safety" level of upgrade. Since damage to the historic fabric could in many cases prove to be a life safety issue, many life safety concerns can be satisfactorily addressed by pursuing rehabilitation objectives that preserve and protect the historic fabric of the building. This is particularly true for areas such as the public exit ways through the grand east and west hallways and the rotunda as well as all other interior and exterior spaces where building components could become falling hazards. Since many of the features comprising the historic fabric and architecture of the building could become life safety falling hazards in these areas, life safety concerns are addressed by pursuing measures which preserve, protect and stabilize the historic fabric against damage and/or falling due to seismic motion.

5. FINDINGS

a. Earlier Findings

For the 1993 evaluation, the following basic structural deficiencies were identified:

- Basic structural framework is very heavy, brittle reinforced concrete frame building with unreinforced masonry and stone infill walls between the concrete frame.
- The concrete frame was designed and constructed to support gravity loads only. The frame does not have the strength to function as an effective seismic force resisting system (Lateral Force Resisting System or LFRS).
- The concrete frame lacks the required amount of reinforcement to behave in a flexible manner.
- The unreinforced masonry and stone infill walls lack the necessary strength to remain intact when subject to significant seismic loads.
- The building has large floor and roof openings and at level 3, 4, the attic level, and the roof. These openings can cause unstable seismic behavior and stress concentrations.

- Noncontinuous infill walls and interior partition walls create stress concentrations which tend to concentrate damage at these locations.
- The concrete vault walls and original interior masonry partition walls are discontinuous from floor to floor. This creates a change in stiffness and tends to concentrate loads at these locations.
- The exterior masonry piers lack the capacity to resist loads that cause uplift (rocking).
- Unbraced, unreinforced masonry parapets and balustrades exist round the perimeter of the roof that could become falling hazards.
- Unreinforced masonry and stone walls have inadequate anchorage to the structure beneath.
- The concrete dome cylinder has a primary weakness (stiffness discontinuity) due to the windows that penetrate the cylinder walls.
- Earthquake forces cause uplift on the footings at the foundation of the dome.
- The concrete columns and walls in the four major piers supporting the reinforced concrete arch beams below the dome cylinder are lightly reinforced and do not have sufficient seismic capacity.

For the 1993 study and evaluation the structure was deemed inadequate in terms of required capacity to resist earthquake induced lateral loading.

b. Discussion of Primary Deficiencies

1. Inadequate Concrete Reinforcement

The items identified as structural deficiencies in the 1993 report are still deemed as deficiencies per current standards and by FEMA 273 guidelines. Although the basic structural component of the facility is reinforced concrete, the amount of reinforcement in the concrete is far less than would be considered minimum for adequate seismic performance and ductility. Ductility is the ability of a structure to experience large levels of lateral movement without collapsing nor losing its ability to support gravity (vertical) loads. For concrete frames, ductility is generally provided by the longitudinal reinforcing steel that is held together (confined) by closed reinforcing stirrups or ties. Confinement provided by ties prevents longitudinal bars from buckling through the column surface and hold the concrete core so that it stays together when cracked. When the longitudinal reinforcement is adequately confined, concrete frame elements can be very ductile and can sustain their load carrying ability even when cracked or deformed. When inadequate longitudinal reinforcement is present and/or inadequate confinement of longitudinal reinforcement is present, the structure is very brittle and does not have the ability to maintain its gravity load carrying capacity under large movements or after significant cracking.

For the Utah State Capitol, the concrete columns are typically reinforced with 4 smooth vertical reinforcing bars (one at each corner) with ¼" diameter ties at about 12" on center (see Figure 6). The longitudinal reinforcement generally has a total cross sectional area that is about 0.4% to 0.6% of the gross cross sectional area of the column. Current standards require that a column (carrying gravity or seismic load) have longitudinal reinforcement equal to at least 1% of the total cross sectional area of the column. Current codes also require tie diameters to be much larger and tie spacing to be much closer than that used in the Utah State Capitol. Many of the reinforcing bars used for the facility, unlike bars used in modern construction, are round smooth or square smooth

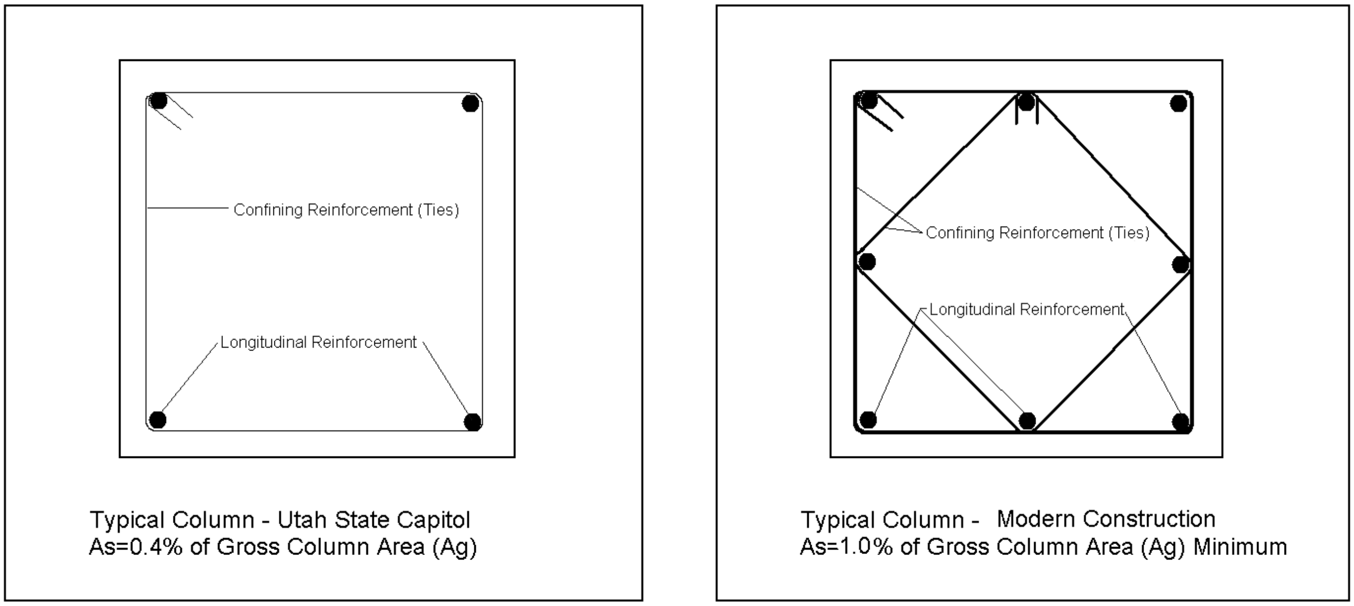


Figure 6 Comparison of Concrete Columns

twisted bars and therefore do not interact, bond, and develop in the concrete in an adequate manner. In addition to the overall lack of necessary seismic reinforcement, the lack of the ability of the reinforcement to remain bonded to the concrete reduces the ability of the overall structure to remain stable during significant seismic activity. Due to the reinforcement characteristics in addition to the overall lack of adequate reinforcement, the Utah State Capitol Building is expected to perform in a very brittle, non-ductile manner when subject to the anticipated seismic event.

2. Concrete Dome

The dome structure of the building is at particular risk. Since the various lateral accelerations that a structure experiences coincide with the height of the structure, the dome of the State Capitol will experience more lateral acceleration and displacement than the structure below (see Figure 7). Testing of concrete samples from the dome structure indicate this to be the weakest portion of all of the concrete in the building. This, coupled with the lack of necessary reinforcement for ductility in conjunction with many penetrations at regular intervals around the dome perimeter, makes the dome extremely vulnerable to damage and possible collapse under the seismic hazard characterized for a Life Safety level of performance.

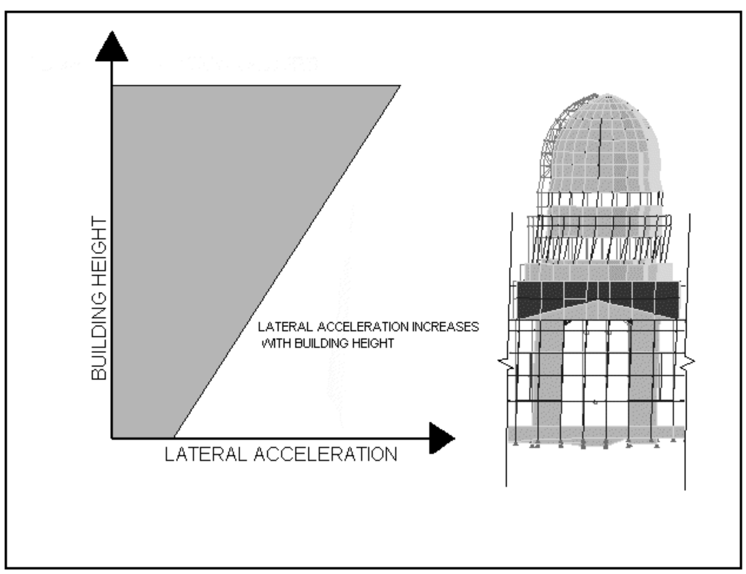


Figure 7 Typical Horizontal Acceleration Profile

c. Basic Structural Dynamics

The results of the SAP2000 computer modeling indicate that the fundamental period of vibration for the structure is approximately 1.1 seconds. The fundamental period is the time it takes for a structure to move through one cycle of natural vibration. The nature of this mode of vibration is torsional which means rather than moving uniformly in any one direction, the entire structure tends to rotate about the core (see Figure 8). This unbalanced mode of vibration is due to the large concrete elements that support the dome in the center of the building. Since these elements are extremely stiff in comparison to the remainder of the structure, the structure tends to pivot about the center of this group of elements. This mode of vibration for a structure is generally avoided in the design of new structures because of its irregularity. While the center of the building might remain stable and might experience minimal displacement during an earthquake, the twisting effect could lead to significant displacements and damage at the extreme east and west ends of the structure. Since the extreme east and west ends of the structure are so far removed from the stiff components that support the dome at the center of the structure, the stiff core is of little benefit to the structure at the east and west ends of the building. At these locations, lateral forces are resisted almost entirely by the limber, lightly reinforced concrete frame. Damage at the east and west ends of the as-is structure due to its torsional behavior is expected to be severe with the possibility of structural collapse due to the anticipated seismic event.

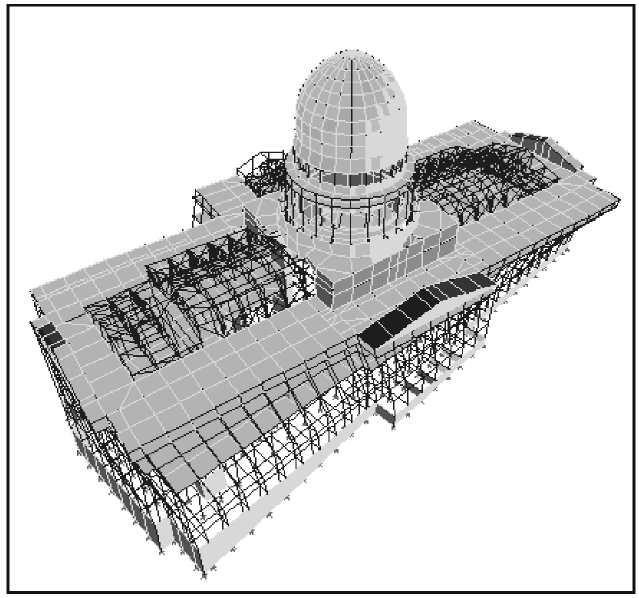


Figure 8 Fundamental Vibration Response Mode

d. Building Redundancy

When most of the seismic load of a structure is concentrated into a single group of relatively few elements the risk of serious damage increases. When one of only few elements supporting the bulk earthquake load fails, the remaining elements are required to support the balance of the load, which could lead to their successive failures. A structure is said to be redundant when it has enough seismic force resisting elements to provide an even distribution of lateral load across its breadth. Newer building codes encourage redundancy by requiring penalties in the form of increased design load when minimum requirements for redundancy are not met. Since the core of the structure resists the bulk of the seismic load, the Utah State Capitol cannot be classified as redundant. It therefore is considered more of a seismic hazard and danger to life in terms of expected seismic performance. In addition, the core does not have the capacity to support the anticipated seismic load nor does is have the ductility to perform well seismically. The expected seismic performance at the core of the structure is very poor with likely brittle failure and possible structural collapse.

e. As-is Building Lateral Displacement Capacities

1. Basic Structure Capacity

For a level of lateral force corresponding to the 10% in 50 year seismic event, the analysis indicates that the as-is structure has a theoretical horizontal elastic displacement of approximately 9 inches at the roof level. For the top of the dome, the horizontal displacement is approximately 26 inches (see Figure 9). Please note that these displacements are based on the elastic (linear) analysis of the structure. This method of analysis does not account for the nonlinear behavior that is certain to occur during a significant earthquake. Nonlinear or inelastic behavior occurs when structural elements reach their maximum elastic capacities and begin to either yield or fail. As the structure begins to behave nonlinearly, the fundamental dynamics are changed and the structure generally begins to respond less dramatically to seismic excitation. Nonlinear behavior can be a useful method of controlling structural response if the structure has been designed to behave in a ductile manner. Methods of nonlinear analysis (pushover analysis) prescribed by NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS (FEMA 273) indicate that the basic structural framework (structure other than the dome and its supporting elements) has the capacity to behave in an elastic manner to a rooftop displacement of approximately 4 inches. At this point the reinforcing steel begins to yield in many of the structural members. At a displacement of 6 inches brittle failure mechanisms begin to occur and portions of the structure could begin to collapse. Since the computed deflection is approximately 9 inches and the computed collapse threshold is only 6 inches, the basic structure of the Utah State Capitol has about 2/3 of the capacity that is needed to resist the level of seismic load corresponding to the characterized Life Safety seismic hazard (10% in 50 year event). For the characterized collapse prevention level of seismic hazard (MCE level event) the calculated maximum rooftop elastic displacement is approximately 26 inches with a total dome displacement of approximately 48 inches, both of which significantly surpass the expected capacity of the basic structure and the dome.

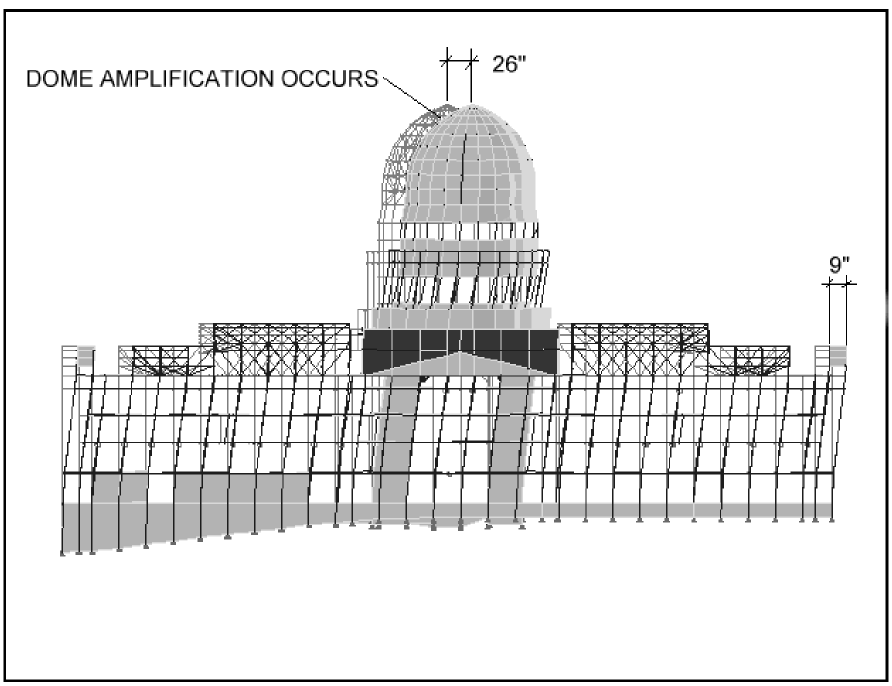


Figure 9 Elastic Displacements Under Loads of 10% in 50 Year Seismic Event

Nonlinear analysis methods prescribed by FEMA 273 enable the calculation of the more realistic nonlinear displacement in conjunction with the calculation of expected displacement capacities. For the 10% in 50 year (Life

Safety level) event, the nonlinear displacement at the roof level of the Capitol Building (sometimes referred to as the target displacement) is expected to be approximately 9 inches for the as-is structure. For the less frequent but more severe MCE event, the nonlinear displacement at the roof level is expected to be approximately 27 inches for the as-is structure. Both of these calculated nonlinear displacements for the anticipated seismic events are well in excess of the calculated structural capacity thus indicating that overall expected seismic performance of the building is very poor and the possibility of structural collapse is high.

2. Dome Capacity

Due to its mass, height and stiffness, the earthquake induced horizontal motions of the dome of the Capitol Building are amplified. The analysis indicates that the dome of the structure is likely to experience at least twice the horizontal acceleration due to earthquake motion than the roof. In addition the concrete compressive test results indicate that concrete strengths for the dome structure are very low. This, in conjunction with the relatively light reinforcement used for the dome concrete increases the likelihood of brittle failures for the dome structure. Use of the analysis methods of FEMA 273 (pushover analyses) indicates that the dome of the Capitol Building has the capacity to displace elastically with respect to the roof approximately 5 inches at which point the reinforcing steel yields and the supporting system of the dome begins to behave inelastically. The analysis indicates that the dome can experience further displacement up to a total of approximately 7 inches before the columns supporting the dome at the windows in the lower cylinder begin to form brittle failure mechanisms that could lead to the collapse of the dome. Under the BSE-1 (10% in 50 year seismic event), the elastic displacement of the dome is approximately 26 inches with respect to the foundation (see Figure 9). This would make the lateral displacement of the dome, relative to the roof, approximately 17 inches. Based on these analysis results, the dome structure has less than half the capacity needed to resist the load of the characterized Life Safety seismic hazard. For the BSE-2 (MCE level event) the elastic displacement of the dome is calculated to be approximately 48 inches relative to the ground. The structure at the lower part of the dome therefore only has a fraction of the capacity needed to perform satisfactorily for the more severe MCE seismic event.

f. Nonstructural Elements and Components

In addition to the poor expected performance of the basic structure, many of the nonstructural components of the as-is building are also not expected to perform well under the characterized seismic hazard. These items include (but are not limited to):

- Exterior appendages with anchorages that are either corroded, inadequate seismically or both. Anchorages of existing cladding and components are believed to be corroded and inadequate seismically. The extent of the damage is not known and cannot be determined without extensive demolition into the existing cladding.

- Cladding which is inadequately anchored that could become a life safety falling hazard at points of egress.
- Parapets around the perimeter of the roof which are not securely anchored and are expected to experience significant seismic motion.
- Exterior (stacked) granite columns on the south, east and west exteriors of the building. Analysis indicates that these columns remain stable up to a horizontal acceleration of about 0.2g (20% of gravity), much less than that expected from the characterized seismic hazard.
- Interior components such as plaster ceilings, marble cladding, and other items that are not adequately anchored to the structure.
- The monumental stairs at the east and west ends of the main hall. Since the stair assemblies are rigidly connected from level to level they could act as very stiff components that attract large amounts of seismic load which they do not have the capacity to support.
- Skylight assemblies. Since the skylights blanket the large floor openings over the main part of the structure, they could experience considerable seismic deformation. They likely do not have the ability to remain intact under seismic load and could therefore become a serious falling hazard to the occupants in points of egress below.

Other nonstructural items typically found in older structures that could also present life safety hazards include:

- Unreinforced masonry partitions and hollow clay tile partitions.
- Major mechanical equipment suspended from ceilings without braces.
- Elevators with counterweights not adequately braced and secured.
- Nonstructural elements that may fall and impede egress.
- Hazardous materials and chemicals.

1. PREVIOUS RECOMMENDATIONS AND ALTERNATIVES

As part of the 1993 report several performance goals were proposed along with several different retrofit options to meet the performance goals. Included also were two options that were examined but not recommended. These options included doing nothing to mitigate the possible damage or loss that may occur due to a significant seismic event or to minimize the building occupancy limit and time of occupancy to reduce the likelihood of people being in the building when a significant seismic event occurs. Neither of these options were recommended in the 1993 report as a viable method of dealing with the seismic issue. The performance goals listed in the 1993 report are as follows:

- PERFORMANCE GOAL A – Strengthen the existing building to reduce collapse potential. This goal is similar to the Collapse Prevention rehabilitation objective defined by FEMA 273.
- PERFORMANCE GOAL B - Strengthen the building structure to a level where structural damage after a major earthquake could be repaired and brace or reinforce selected nonstructural elements for increased life safety as well as those of historical importance. This goal is similar to the level of performance falling between a Life Safety and a Collapse Prevention level of performance defined by FEMA 273 as a Limited Safety Range of performance.
- PERFORMANCE GOAL C – Strengthen the building according to performance goal B and brace all feasible architectural, mechanical, and electrical elements. This would provide increased protection to life and would be similar to the Life Safety level of performance defined by FEMA 273.
- PERFORMANCE GOAL D – Improve the seismic resistance of the building structure and brace the nonstructural elements so only minor disruption to the building is experienced after a major earthquake. With this performance goal a base isolation system is recommended. This level of performance is similar in nature to the Enhanced Rehabilitation Objective defined by FEMA 273. For this objective, it is desired not only to prevent loss of life due to seismic activity, but to take measures to limit the amount of damage to the structure and its components.

Of these performance goals, only goals C and D are in accordance with the minimum recommendations per FEMA 273 guidelines. The recommended Basic Safety Objective would not be achieved with performance goals A and B from the 1993 report.

2. RECOMMENDED REHABILITATION OBJECTIVE

The primary objective of this evaluation is to recommend methods to seismically rehabilitate the Utah State Capitol to ensure life safety of the occupants. However, due to the historic nature of the Utah State Capitol and the irreplaceable components of the facility, an Enhanced Rehabilitation Objective at minimal added cost is recommended for the analysis and retrofit of the facility. As a minimum, this rehabilitation objective should require that the building perform to a Life Safety (LS) level for the BSE-1 (10% in 50 year) earthquake and Collapse Prevention (CP) for the BSE-2 (MCE or 2% in 50 year) earthquake. As an enhanced objective, rehabilitation measures should be chosen which will prevent the historic fabric of the building from becoming life threatening falling hazards, thus life safety concerns would be met while simultaneously providing a level of performance that preserves the elements and components of the building.

3. SUMMARY OF POSSIBLE RETROFIT SCHEMES

The following are retrofit schemes that have been considered as possible recommendations to bring the Capitol Building to a minimum level of performance to meet the recommended rehabilitation objective:

- Add diagonal steel bracing to increase overall structural stiffness. The steel bracing would add increased structural stiffness and significant seismic strength to the structure.
- Add a system of passive dampers to absorb earthquake energy to reduce overall seismic response. The dampers would act as shock absorbers which would collect the energy from and earthquake and dissipate it in a safe manner rather than allowing it to damage the structure.
- Add reinforced concrete moment frames to increase overall stiffness and to improve ductility. Since the as-is structure lacks the overall ductility to perform well seismically, the addition of ductile concrete moment frames could provide the ductility to make the structure perform adequately when subject to the expected seismic hazard.
- Add concrete shear walls to increase overall structural stiffness. The configuration of concrete shearwalls that could be added can vary significantly. The added shearwalls could provide significant seismic strength and could bring the structure to and acceptable level of performance per FEMA 273 guidelines.
- Add a base isolation system to reduce the overall amount of seismic demand on the structure. A base isolation system would 'filter' the ground motions and reduce them to a point that the structure responds less dramatically with a much lower level of overall acceleration under the expected seismic hazard.

Each of these schemes has been evaluated in great detail to determine which will be the most effective for meeting the recommended rehabilitation objective. Although each option could be made to work, some are better than others in terms of cost, function and effectiveness.

The addition of steel braces is not recommended as this system would require an extreme number of braces to be effective. The analysis has shown that to resist the characterized seismic loads, braces would need to be added to an unusually large number of bays within the structure. This would require significant intrusion into the existing finishes and would likely interrupt functional spaces in addition to the historic fabric. Although the braces could provide added stiffness that would significantly improve the expected seismic performance of the building, they could not be incorporated into the existing structure as readily as other systems. Effectively connecting new structural steel braces to the existing concrete would be a challenge. Since the seismic loads in the braces would be very large, the connections would require special considerations and unusual detailing which would inevitably increase the cost of this system. Steel braces are also incompatible dynamically with the existing system and would not meet the recommended rehabilitation objective as effectively or economically as other systems.

Passive dampers installed in a manner similar to diagonal bracing could be used as a system which would absorb the energy from an earthquake and dissipate it in a safe manner rather than allowing it to become manifest as structural damage. Dampers added to the Capitol Building would act as "shock absorbers" that have the ability to dampen the impact of a significant seismic event. However, passive dampers are a velocity dependent mechanism that would require the structure to become mobile and deform before becoming effective. For the dampers to absorb and safely dissipate energy, the structure must shake and deform horizontally. This movement enables the dampers to engage in reducing seismic load which then results in less movement and less earthquake induced damage. The analysis of the as-is structure indicates a high likelihood of brittle failure for small movements and structural deformations. Since small movements and deformations could cause life threatening structural instabilities, passive dampers are deemed less effective than other available systems. Although a passive dampening system could be made to work, it would not readily meet the recommended rehabilitation objective as would other systems in terms of cost and function.

Concrete moment frames could be added to enhance the overall ductility for the structure. The new frames would encase the existing frames to become the new primary seismic load resisting system (Lateral Force Resisting System or LFRS). However, moment frames are inherently limber and are generally expected to deform significantly under design level seismic loading. Significant deformation could be extremely detrimental to the existing brittle concrete framework of this structure and could pose life safety hazards. In addition, analysis indicates that for concrete moment frames to become effective for this building, they would need to be very large and a significant number of frames would need to be added. It is likely that for this scheme to work, many of the existing finishes and cladding would need to be removed and modified to accommodate this system. In addition, the system would likely reduce or interfere with many of the functional spaces. Although a reinforced concrete moment frame could be made to work, it is not the most effective in terms of cost and function.

Based on the analyses and findings for each of the considered retrofit schemes, a base isolation system, a fixed base shearwall system or a combination of base isolation and shearwalls are the most effective alternatives for meeting the recommended rehabilitation objective. These more effective options have been analyzed in great detail, the results are herein provided.

a. Proposed Shearwalls for Fixed Base Shearwall Option

The addition of shearwalls to the structure could greatly enhance the ability of the structure to resist torsion and to distribute the seismic load evenly across the breadth of the structure. Placing transverse shear walls at the east and west sides of the structure (Figure 10) can reduce the torsional effect as a primary mode of vibration. This will reduce the potentially large displacements and possible collapse at the extreme east and west ends of the structure. The shearwalls will also resist a large portion of the seismic load that could act in the north-south direction. Using the shearwalls as shown will also decrease the amount of concentrated earthquake load in the floors since the loads are more evenly distributed to the new shearwalls located at regular intervals across breadth of the building. Longitudinal shearwalls placed along the interior corridors will also provide a better distribution of earthquake floor loads for loads acting in the east-west direction. New perimeter shearwalls will significantly stabilize the building against torsional vibration and provide increased rigidity to the structure as a whole. The addition of shearwalls as shown can significantly reduce the amount of lateral deformation that the structure could experience during a significant seismic event. This would result in decreased demand on the existing structural members that are incapable of experiencing even moderate deformations without brittle failure.

The shearwall option shown in Figure 10 is similar to an option proposed in the 1993 study. Unlike the analysis performed in 1993, the current study includes the extended three dimensional finite element computer analysis of this shearwall option. As part of the current study, the shearwall configuration shown in Figure 10 has been added to the analytical computer model of the as-is structure. This has enabled an evaluation of this shearwall scheme that is far more complex and descriptive than that performed in 1993.

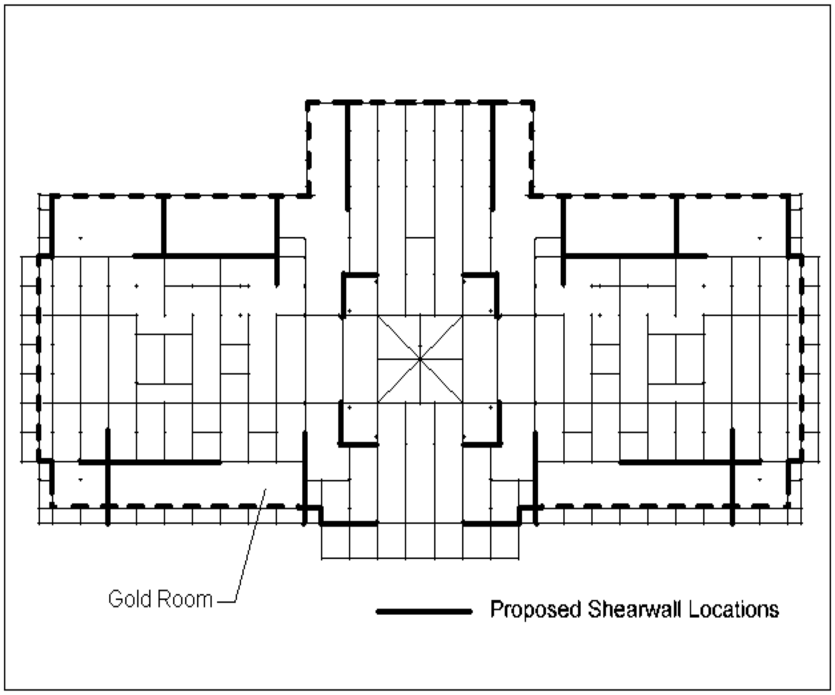


Figure 10 Proposed Shearwalls for Fixed Base System

It is expected that the fixed base shearwall system shown in Figures 10 and 11 could stand alone as a fixed base primary system for resisting earthquake induced horizontal loads. For this option, the new structural shearwalls added to the perimeter of the structure on each level along with the new interior shearwalls as shown would meet the recommended rehabilitation objective and would provide a structural system capable of withstanding the seismic motions corresponding to the characteristic seismic hazard. For this option, the new perimeter as well as the new interior shearwalls could be added with either a formed or a sprayed concrete application. For the new perimeter shearwalls, the new concrete would effectively fill the space behind the cladding which now is filled with hollow clay tile and/or brick masonry. It would be applied to the sides and below each window opening and would attached to each existing perimeter concrete column. As an added benefit, this added reinforced concrete would provide a method of anchoring the granite cladding on the exterior of the structure to prevent it from falling due to earthquake motion.

For the fixed base shearwall option shown in Figure 10, the analysis indicates that the total horizontal drift (displacement) at the roof level of the structure corresponding to the MCE seismic hazard could be reduced from a worst case of 26 inches to approximately 4 inches (see Figure 11) thus reducing the drifts to a level at which the existing basic structural framework remains stable. Since the existing structural components would not experience displacements and rotations as high as the unreinforced as-is structure, they would remain more capable of supporting load thus leading to a more safe structure with less damage caused by the anticipated seismic event.

This shearwall option is also considered to be quite flexible in terms of placement and accommodating the existing historic fabric of the building. Though a symmetrical layout of walls is the best configuration, the layout of walls as shown in Figure 10 could be adjusted to accommodate existing

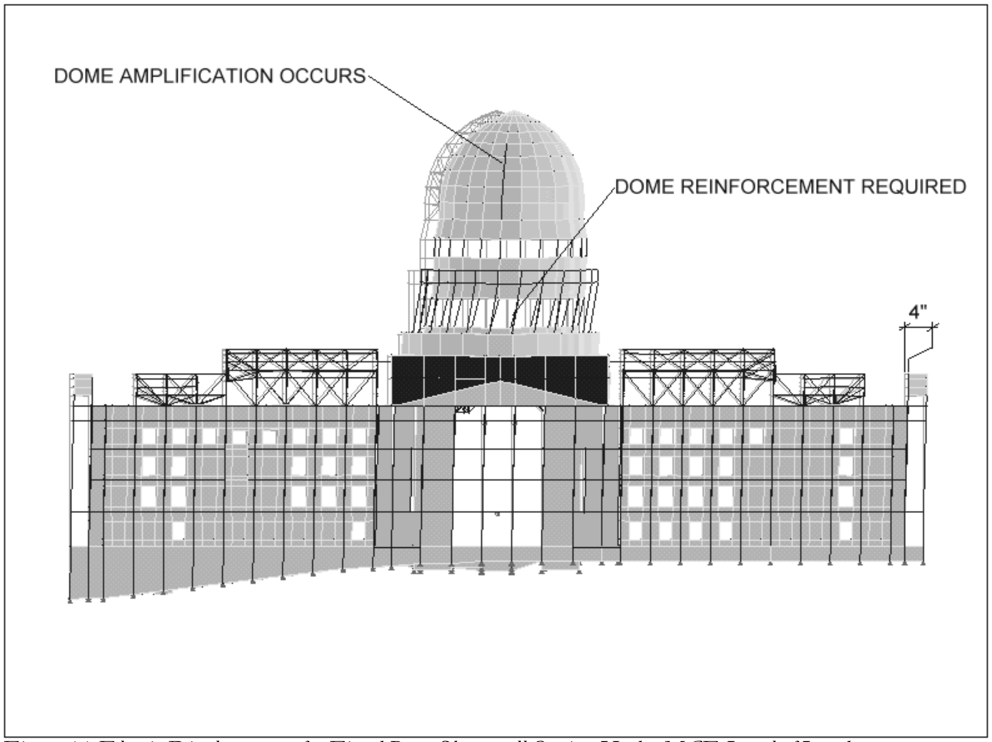


Figure 11 Elastic Displacements for Fixed Base Shearwall Option Under MCE Level of Load

historic finishes while still providing the same amount of upgraded strength. For instance, at the Gold Room it may be desired to avoid demolition of finishes that could be very costly or difficult to replace. At this location the proposed shearwalls could be moved to the next bay space away (typically 14') thus preventing intrusion of the significant historic space while still gaining the added strength of the shearwalls. Preservation of the exterior walls within the Gold Room could also be achieved by simply omitting the new shearwalls at this one location. Although the omission of proposed shearwalls is not usually a good approach, it would not significantly change the overall structural behavior if done sparingly at this one location. Since the exterior cladding at this area would likely fall in a location of restricted access, failure to adequately anchor the exterior cladding just outside the Gold Room to the structure would not constitute a serious life safety concern.

Figure 12 depicts a second alternative for a shearwall retrofit scheme. Though this method of adding shearwalls is far less intrusive in terms of the building's historic fabric, it does not provide the optimal layout and expected seismic performance that would be achieved with the first option (Figure 10). Since this option is considered less effective as a fixed base scheme, it is recommended for use in conjunction with a base isolation system. Refer to Appendix drawings for plans and details of this configuration. Similarly, the shearwall

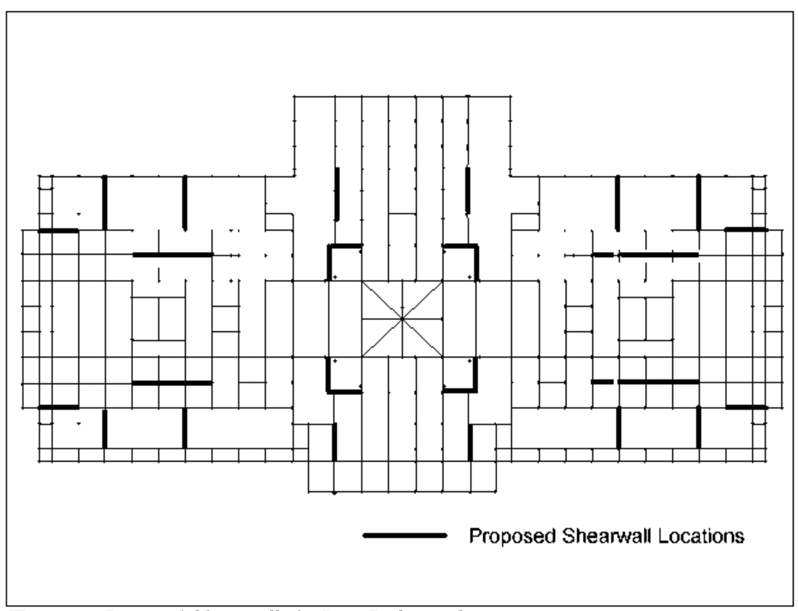


Figure 12 Proposed Shearwalls for Base Isolation Option

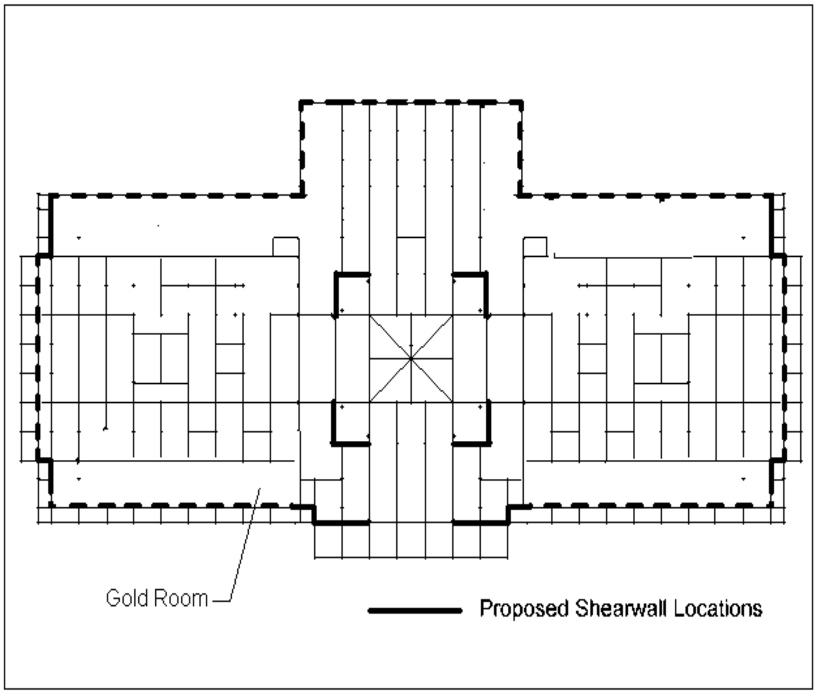


Figure 13 Proposed Shearwalls for Base Isolation Option

layout shown in Figure 13 could be made to use in conjunction with a base isolation system. For this system, a number of the interior shearwalls could be eliminated and replaced by perimeter shearwalls similar to those shown in Figure 10. Analysis shows that this system is could behave in a manner similar to the configuration shown in Figure 12 with the added advantage of bracing the exterior cladding so that it does not become an earthquake falling hazard.

For all of the shearwall options shown, reinforced concrete shearwalls should be added to the triangular shaped concrete piers that support the dome structure (see Appendix drawings and details). The existing piers are very rigid and will thus attract a large part of the structure's seismic load. However, the piers are not adequately reinforced to behave in a ductile manner and therefore could be subject to brittle failure mechanisms when subject to the characterized seismic event. New reinforced concrete could be added to the inner (hollow) portion of the piers which would greatly improve the ability of the piers to perform in a safe, ductile manner.

For shearwalls to effectively collect load and distribute it to the foundation of the structure they must extend from floor to floor on each level from the basement to the roof and must be securely anchored to the floor diaphragm at each level. For the floor system to effectively transfer seismic load into the shearwalls, drag struts must be added as required to collect load from each diaphragm (floor) segment and deliver it to the concrete shearwalls (see Appendix drawing S-3 for conceptual layout and configuration of drag struts in addition to detail 5/S-6 for drag strut configuration). Drag struts extend horizontally through the floor from the edge of a shear wall. The struts are designed to collect the earthquake load in the floor which they engage so that it does not become a concentrated load at the edge of the shearwall.

Although the addition of structural shearwalls of the fixed base system would significantly improve the overall expected seismic performance of the structure, their addition could be detrimental to many of the nonstructural components of the building. When shearwalls are added to a structure, it becomes considerably more stiff and the primary periods of vibration are reduced. The computer modeling and analysis of the fixed base shearwall scheme has shown that the period of the fundamental mode of vibration changes from 1.1 seconds to 0.6 seconds. The nature of this mode

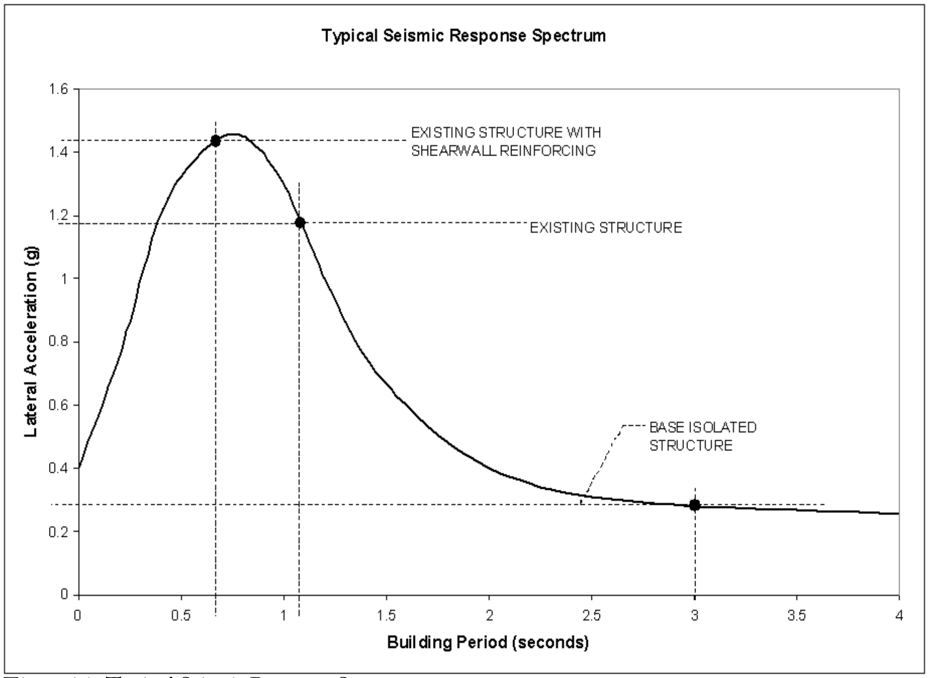


Figure 14 Typical Seismic Response Spectrum

of vibration is orthogonal which means the structure moves uniformly in one direction rather than twisting in torsion. Since the fundamental period has changed from 1.1 second to 0.6 seconds, the structure is significantly more rigid. As a result, the overall structure experiences far less deformation but more lateral acceleration than the original unmodified structure (see Figure 14). If the fixed base shearwall option of retrofit is selected for the rehabilitation of the Capitol Building, it will require very careful consideration of the adequacy (and possible deterioration) of the anchorage of all of the nonstructural components of the building. Each nonstructural item that could become a life threatening falling hazard must be carefully studied and braced to address life safety concerns. In addition, the amplification of seismic load of the dome structure would be dramatically increased if the shearwall only option is chosen for retrofit. Studies have shown that dome amplifications can be dramatic for structures with configurations similar to the Utah State Capitol.[6] Like many of the nonstructural components of the building, the dome would experience higher levels of lateral acceleration and thus higher seismic loads if shearwalls alone are used in the retrofit. This would therefore require more extensive retrofitting and modification of the dome structure to bring it to an appropriate level of expected seismic performance. The dome accelerations for the fixed base shearwall option are manifest as significant dome displacement. Figure 11 shows the fixed base shearwall scheme with very little relative displacement at the roof level of the structure (4 inches). However, the dome is still subject to considerable movement and horizontal forces. Despite the addition of significant concrete reinforcing elements to the dome and its supporting structure, the computer modeling indicates that dome displacements are still approximately 16 inches with respect to the ground, thus indicating that extensive retrofit measures must be pursued at the dome of the structure if the fixed base shearwall option is chosen as the sole method of strengthening and retrofitting this structure.

Despite the increased horizontal accelerations, the fixed based shearwall retrofit scheme could be made to work as an effective retrofit method of bringing the Capitol Building to and adequate level of expected seismic performance. The recommended rehabilitation objectives would be met and the threat to life would be minimized.

6. REPORT, *Seismic Protection of Domed Structures*, Forell/Elsesser Engineers Inc., San Francisco, CA.

b. Proposed Base Isolation System

Although the fixed base shearwall scheme could be made to work, it would require extensive retrofitting of the structure supporting the dome in addition to many of the nonstructural elements and components of the building. It could be very intrusive to the fabric of the building and would require extreme measures and costs to brace all of the components that could become life threatening falling hazards. For these reasons a base isolation system is recommended in conjunction with a minimum amount of new shearwalls to lower the level of expected structural movement. A base isolation system would reduce the horizontal movements of the structure thus minimizing the amount of retrofitting required to bring the building to an adequate level of expected seismic performance.

Seismic base isolators placed at the foundation level of the structure act as lateral floating mechanisms that filter and reduce the ground accelerations that result from an earthquake. Although seismic base isolators do not completely eliminate the horizontal earthquake forces that a structure could experience, they can significantly reduce the seismic forces to which a building is subject. The concept of base isolation is simple. Engineers and researchers have long understood that limber structures with relatively long fundamental periods of vibration respond less dramatically to lateral seismic motion than do stiff structures with relatively low fundamental periods. The fundamental period is the time it takes for a structure to move through one cycle of natural vibration. For example, a short flagpole vibrates faster than a tall flagpole, it is therefore subject to greater levels of horizontal acceleration. Structures behave in a similar manner. Tall or limber structures vibrate slowly and thus do not respond as dramatically as short/stiff structures to earthquake ground motions. Short or stiff structures vibrate faster and therefore respond with increased horizontal accelerations due to earthquake ground motion. Although limber structures respond with less horizontal acceleration to earthquake ground motion their overall horizontal displacements are more because they are limber.

1. Typical Base Isolated Building Behavior

Base isolation enables a structure to behave as a limber system while maintaining all of the advantages inherent in a stiff structural system. Base isolation increases the fundamental period of a structure so that it responds less dramatically to seismic activity. Figure 14 is a representation of a typical seismic response spectrum. A seismic response spectrum can be used as a measure of a structure's expected earthquake response. The horizontal axis represents the fundamental period of the structure and the vertical axis represents the amount of seismic load (acceleration) that the structure could experience in an earthquake. Note that if the fundamental period increases, the spectral acceleration response of the structure typically decreases. Since the global system (structure and base isolators) is a limber system, the total displacements of the top of the structure relative to the ground are very large. However, it should be noted that the bulk of this displacement is experienced by the base isolation system itself leaving very little relative displacement in the structural system above the isolators. The resulting lateral accelerations and relative displacements of the structure above can be significantly reduced thus protecting the occupants and preserving the structure along with its contents.

2. Components of an Isolation System

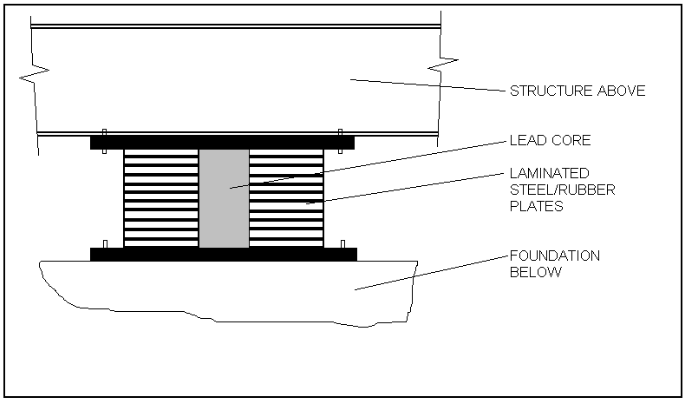


Figure 15 Elastomeric Base Isolator

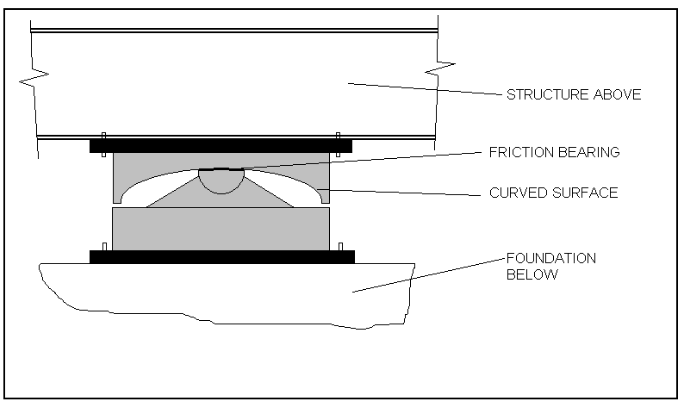


Figure 16 Friction Pendulum Base Isolator

A base isolator is defined as horizontally flexible and vertically stiff structural element that permits large lateral deformations under seismic load. An isolation system is defined as a collection of isolator units tied together by a structural diaphragm that enables the group of isolators to act collectively to alter the fundamental dynamic characteristics of the structure above. Figure 15 is a schematic representation of a typical elastomeric laminated rubber and steel base isolator unit. This isolator is typically composed of multiple circular or square layers of laminated rubber and steel plates and can also have a core of lead that provides an additional damping component which absorbs earthquake energy and reduces overall displacement. This isolator can also have external dampening mechanisms that reduce the overall seismic displacement response. A second type of isolator shown in Figure 16 is the friction pendulum isolator. This isolator acts as either a spherical or conical dish that allows large lateral displacements due to the sliding action of the bearing surface on the dish. Damping (energy absorption) with this isolator is provided by the sliding friction surface between the bearing surface and the dish portion of the isolator unit.

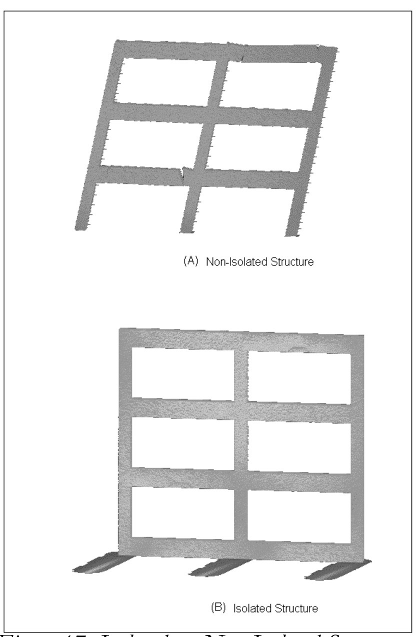


Figure 17 Isolated vs. Non-Isolated Structure (Courtesy, Applied Structures Inc.)

Figure 17 provides a conceptual comparison of an isolated structure and a non-isolated structure. Note that for the isolated structure, the majority of seismic deformation is experienced by the isolation system itself leaving the structure above relatively undamaged. For the non-isolated structure the lateral ground accelerations are transferred through the foundation system into the structure ultimately resulting in the stress and possible failure of structural elements. For seismically isolated structures, it is not uncommon for the level of seismic load to be only 1/5 to 1/6 that of the equivalent non-isolated structure.

3. Anticipated Behavior of Isolated System

Using isolators such as those shown in Figures 15 and 16 can significantly reduce the amount of horizontal acceleration experienced by the structure. The SAP2000 computer analysis of the Utah State Capitol indicates that a seismic base isolation system could reduce the

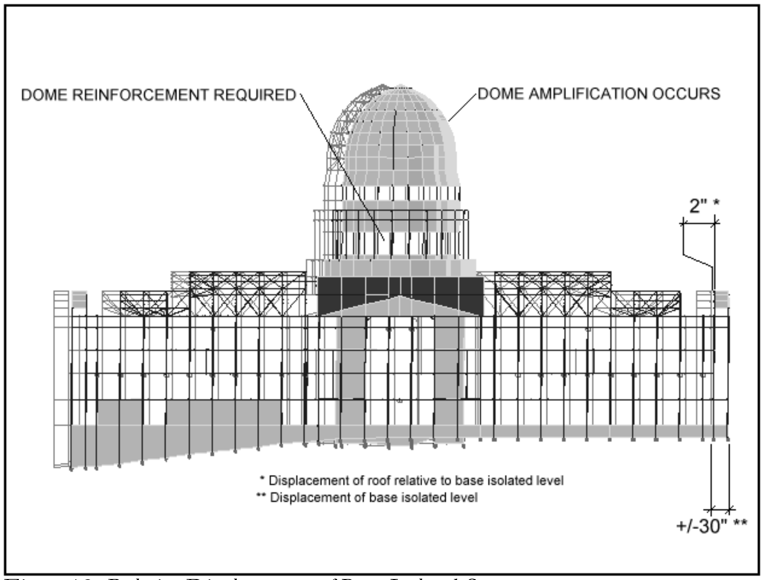


Figure 18 Relative Displacements of Base Isolated System

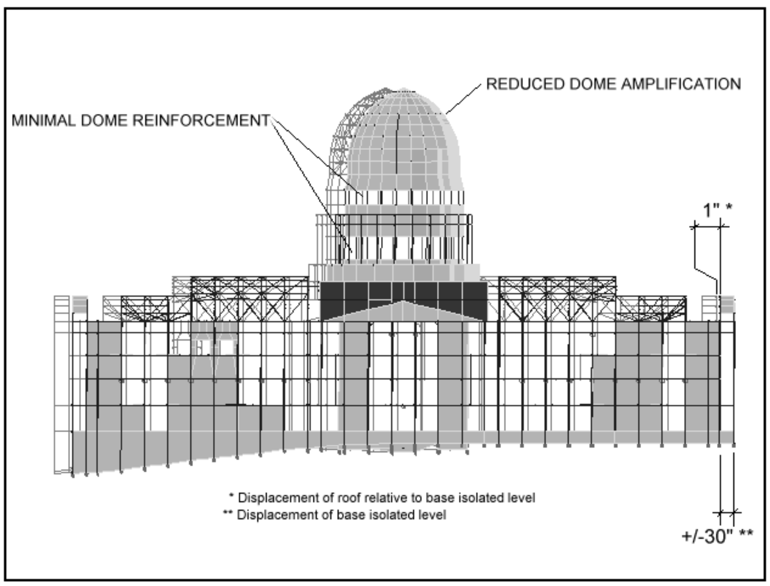


Figure 19 Relative Displacements of Base Isolated System with Shearwalls

amount of lateral load corresponding to the Life Safety level earthquake to approximately one fifth that of the original (non-isolated) structure. It is not uncommon for a base isolation system to experience a lateral displacement (within the system itself) of 30 inches for the design level earthquake. Using a theoretical base isolation system that can displace in any horizontal direction approximately 30 inches results in a significant reduction in force level, horizontal displacement, and lateral acceleration for the Utah State Capitol. Analysis indicates that the base isolation system used without the shearwall combination limits the rooftop displacement to approximately 2 inches for the anticipated seismic hazard (see Figure 18). The analysis indicates that with a base isolation system, in conjunction with the shearwalls recommended previously brings the theoretical elastic displacement of the roof (relative to the isolation system) to approximately 1 inch (see Figure 19), which is far less than that indicated by the analysis of the original, existing structure (see Figure 9). This reduction in relative roof displacement significantly reduces the seismic demand on the structural components. It also reduces the level of nonlinear behavior that the structure would experience corresponding to horizontal loads of the Basic Safety Objective.

A base isolation system is very flexible in terms being able to design a retrofit scheme that is 'tuned' to result in minimal seismic response. In addition, using a base isolation system in conjunction with a minimal amount of new shearwalls would enable tuning of the Capitol Dome structure for minimized dome seismic response. It is expected that a reduction in dome amplification of approximately 40 to 50 percent can be achieved with a base isolation system. Table 2 shows a relative comparison of the reduction in dome amplifications that are expected with the successful integration of a base isolated system. Several other domed structures that have been or are now in retrofit design are included for comparative purposes. Although the base isolation system can significantly reduce the seismic demand of structural members and the overall seismic load, it can be difficult to control amplification of seismic motion of irregular structural systems when used with a base isolation system, such as the dome of the Capitol Building. The isolation system alone cannot reliably prevent amplifications that could lead to significant damage and collapse of the dome and its supporting structure. For this reason it is recommended that shearwalls (mentioned previously) be used in conjunction with the proposed

Structure	Dome Amplification*	
	Fixed Base	Isolated Base
Utah State Capitol	5x	FP=4x Elast.=3x
San Francisco City Hall	3x	Elast.=1.5x
Pasadena City Hall	5x	FP=4x Elast.=2.5x
Oakland City Hall	5x	Elast.=2x

FP=Friction Pendulum Base Isolators
Elast.=Elastomeric Base Isolators
* Amplification of horizontal acceleration as compared to accelerations at ground level.

Table 2 Summary of Amplification of Horizontal Accelerations for Domes of Various Structures for Fixed Base and Isolated Systems.

base isolation system. The shearwalls will enable the design and "tuning" of a retrofit scheme that will minimize the amount of amplification on the dome structure thus minimizing the amount of retrofit that may be required to bring the dome to an acceptable level of seismic performance.

4. Integration of Base Isolation System

To successfully install a system of base isolation on the Utah State Capitol Building it will become necessary to install a new foundation system. The new foundation system will begin with the elimination of all foundation components that are not supporting load. This would include all of the slabs on grade and interior partition walls at the basement level of the Capitol Building. The next step is to excavate around the existing foundation to install a new continuous concrete mat footing that fills in all of the spaces between the existing columns and footings. It is anticipated that the mat footing will be 36" thick. After installing the mat footing, each column or group of columns is successively shored and detached from the existing footing below. The existing footing is then removed to a depth necessary to install a new concrete pier, base isolator jacks, base isolator floor system and accessories. After installing the subcomponents of the isolation system,

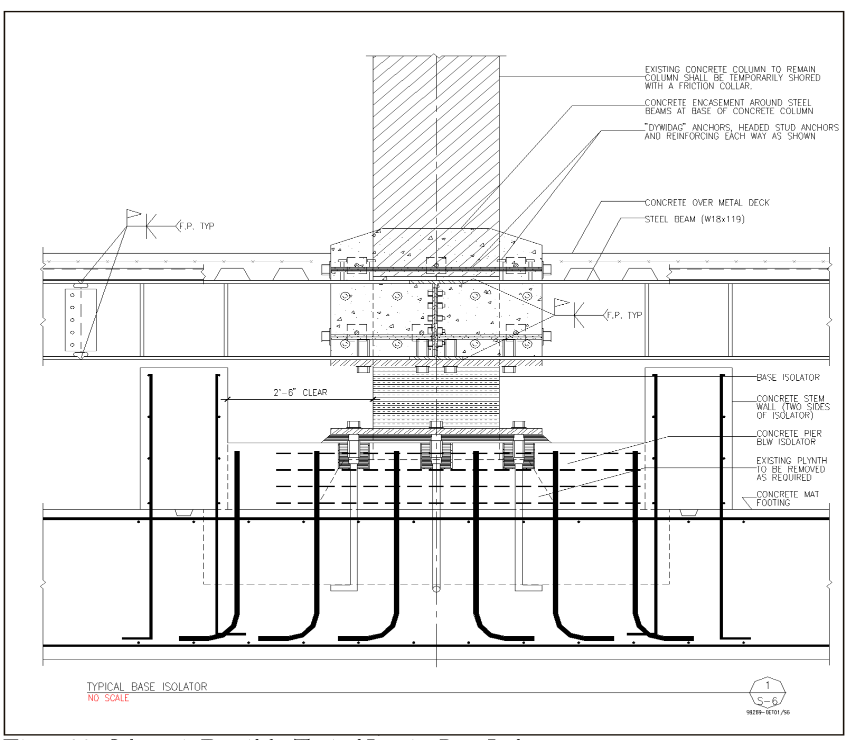


Figure 20 Schematic Detail for Typical Interior Base Isolator

the floor diaphragm system is added above the base isolation system which typically consists of concrete on metal deck with composite steel wide flange shapes. Upon substantial completion of the additional necessary retrofitting, the base isolators can be installed and the temporary supporting jacks can be removed. Figure 20 shows the conceptual detailing of a typical base isolation and foundation detail.

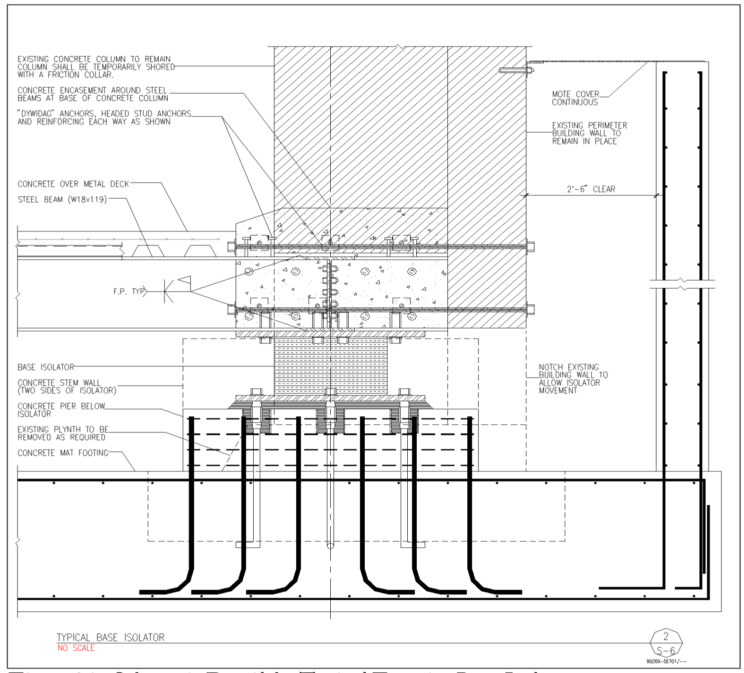


Figure 21 Schematic Detail for Typical Exterior Base Isolator

For the Utah State Capitol Building, it is anticipated that the base isolation floor system will be at a distance of at least 9 feet below the bottom of the Ground Floor framing. The top of the mat footing will be approximately 4 feet below the isolation floor system. For either the laminated steel/rubber or friction pendulum isolator, a space of relief would be required around the perimeter of the structure. Often referred to as a 'mote' this space would be used as a separation of the structure from the adjacent soil thus 'isolating' it from its surroundings (see Figure 21). This space would accommodate the maximum design displacement for the isolation system which, based on the current modeling would be approximately 30 inches. An additional consideration for the base isolation system is the utilities that service the facility. Any piping or conduit that is required to cross the mote must be flexible and must remain intact and functional as the isolation system moves back and forth.

Plans and details for integrating the proposed base isolation scheme are included in the Appendix of this report.

c. Summary of Elastic Rooftop Displacements

Table 3 summarizes the elastic displacements at the roof level for each of the proposed retrofit schemes based on the MCE level of load. Based on the analyses summarized previously, the basic structure for the as-is Capitol Building has the capacity (per FEMA 273 guidelines) at the roof level to move horizontally up to 4 inches before experiencing permanent deformations. The structure can move up to a total of 6 inches before significant damage and possible collapse becomes likely. The retrofit options shown in Table 3 reduce displacements to a level at which the expected seismic performance is greatly improved and the recommended rehabilitation objectives are met.

Building Retrofit Scheme	Approximate Rooftop Displacement Relative to Basement (inches)
Existing building	~26" (worst case)
Shear Wall Model	4"
Base Isolated Model	2"
Shear Wall and Base Isolated Model	1"
Elastic Behavior Threshold*	4"
Generalized Damage Threshold **	6"

Table 3 Summary of Elastic Rooftop Displacements for MCE Level of Load

* This represents the rooftop displacement at which concrete reinforcing bars yield and the structure begins to behave inelastically.

**This represents the maximum rooftop displacement at which serious and/or brittle failure mechanisms begin to occur.

d. Advantages and Disadvantages of Proposed Retrofit Schemes

1. Fixed Base Shearwall Option

The following are the primary advantages of using the fixed base shearwall retrofit scheme:

- Provides significant added stiffness and strength.
- Reduction in overall seismic response displacement.
- Reduction in seismic demand on existing structural components due to decreased displacement.
- Use of conventional materials and methods of retrofit and construction.
- Flexibility in terms of locating and placing new shear walls.
- Meets minimum Life Safety and Collapse Prevention rehabilitation objectives per FEMA 273.

The primary disadvantages of the fixed base shearwall scheme include:

- Shearwalls provide increased stiffness which means increased seismic response and higher seismic loads on the overall structure. The loads would be resisted almost entirely by the new shearwalls and not by the existing structure.
- The shearwall only scheme leads to an increase rather than a reduction of seismic response for the dome thus requiring greater retrofit measures for the dome.
- Shearwalls only will increase the expected seismic load on all of the nonstructural elements and components thus requiring more extensive retrofit measures.
- With shearwalls only, more walls would be required thus a greater likelihood of intrusion of the functional spaces and possibly the historic fabric.
- New footings and foundation system would be required.

2. Base Isolation with Shearwalls Option

The following are the primary advantages of the base isolation system used in conjunction with a minimum amount of new shear walls:

- Significant reduction in seismic response with considerably less overall seismic load.
- Fewer new shearwalls above the foundation leading to minimal intrusion into the building exterior and interior above the foundation to install retrofit elements.
- Considerable reduction in dome amplification leading to less extensive retrofit of dome and structure supporting the dome.

- Significant reduction of load on nonstructural components thus enabling minimal retrofit of all nonstructural elements and components.
- Base isolation with minimal shearwalls enables "tuning" for optimal seismic response behavior.
- Exceeds minimum rehabilitation objectives by providing better expected seismic performance due to decreased seismic load.
- Cost of retrofit is significantly less than fixed base system in terms of bracing elements and components that could become falling hazards.
- Preserves historic fabric better than other retrofit options.

The disadvantages of the base isolation system include:

- Use of specialized materials and means of construction.
- New foundation system is required along with significant re-configuration of basement.
- Requires specialized connections for services and utilities to cross the "mote".
- Significant overall lateral deflections require re-configuration of landscaping and site features in the immediate vicinity of the perimeter of the building.

Although either option of retrofit could be used to meet the recommended performance objective, the base isolation system with minimal new shearwalls appears to be the most effective option for achieving the recommended performance objective in terms of both cost and function.

e. Recommendations for Dome

Because of the structural dynamics, lateral seismic amplification and questionable concrete strength at the dome, it is recommended that the concrete members of the dome be reinforced. It is recommended that the dome be reinforced regardless of the specific retrofit option chosen in order to prevent serious damage or collapse caused by a significant earthquake.

It is recommended that the concrete columns supporting the cylinder of the dome structure be strengthened. This can be accomplished by doweling into the existing concrete with new reinforcing bars and casting new reinforced concrete columns at the inside or outside face of the existing columns. Furthermore, it is recommended that every other opening on the upper cylinder of the dome be eliminated to accommodate additional concrete at this elevation. The additional concrete would add a significant amount of strength to the base of the dome and would not significantly affect the exterior or interior appearance and would still allow for a large amount of natural light. Figures 22 and 23 present schematic details for adding the reinforcing elements at the windows of the dome structure (see also 6,7/S-7 in Appendix drawings).

In addition to reinforcing the existing columns, a reinforced concrete ring should be added at the base of the dome cylinders. This reinforcing ring would effectively tie the base of the dome together and make it behave more as a rigid body (see details 6/S-4 and 4,5/S-7 in Appendix drawings).

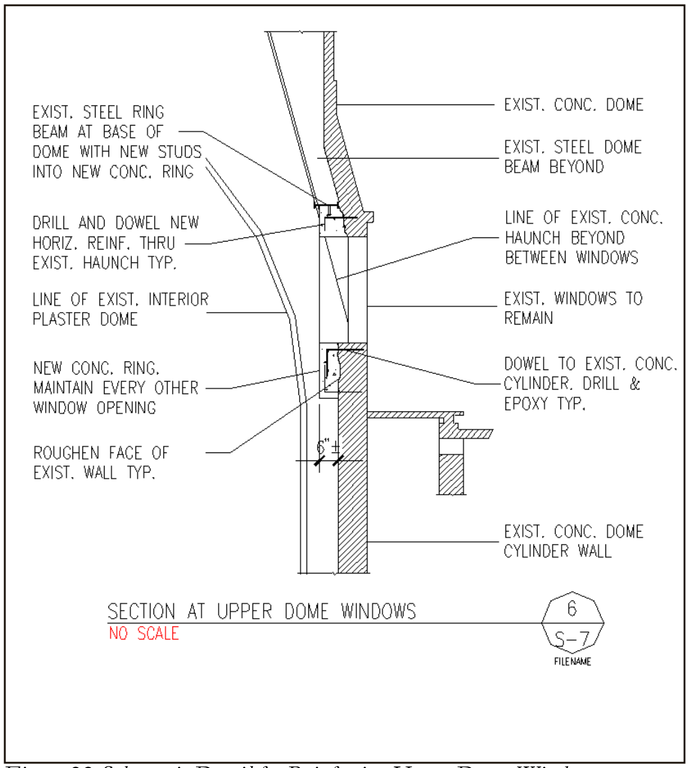


Figure 22 Schematic Detail for Reinforcing Upper Dome Windows

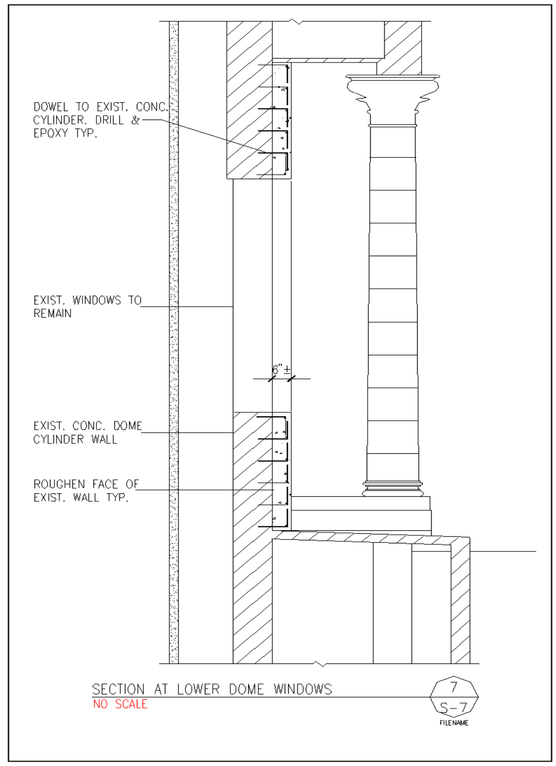


Figure 23 Schematic Detail for Reinforcing Lower Dome Windows

As with other parts of the structure, further investigation of the dome and analysis of alternatives will be required during the next phases of the project to determine the feasibility of adding the proposed retrofit elements.

f. Recommendations for Nonstructural Items

The following items have been identified as nonstructural components of the building that require modification to improve their expected seismic performance.

1. Appendages

It is believed that significant corrosion and other damage to anchorages of stone appendages at the exterior of the structure has occurred. The extent of damage is not known and cannot be determined without extensive demolition into the existing cladding to determine the amount of damage. Further investigation of these elements is recommended during the next phases of the project. Although the capacity of the anchorages of these components is in question, it is probably adequate to maintain the components under gravity load conditions. The retrofit option that may be most beneficial to these components is the base isolation system. By base isolating the structure the lateral accelerations are reduced and thus the lateral forces are reduced significantly. In lieu of providing or replacing anchorage systems to these exterior stone components, the base isolation system may bring the lateral force to a level at which the current anchorages perform to the desired level. At areas of egress all nonstructural building components should be anchored regardless of the retrofit scheme chosen. Falling hazards in these areas represent a threat to life and thus would not meet the minimum rehabilitation objective.

2. Cladding

Although the structure may be strengthened and base isolated many of the contents or nonstructural elements could still sustain damage even under a small level of load if not adequately braced. The exterior cladding of the structure could experience severe damage or could even become a falling hazard if not appropriately anchored. For the exterior walls measures should be taken, if perimeter concrete shearwalls are not added, to anchor or brace the wall components for out-of-plane seismic loads (loads that cause the cladding to fall away from the building). Anchoring the exterior wall with steel columns appears to be an effective option. Detail 1,3,4/S-8 in the Appendix show the recommended method for bracing the cladding to preventing it from becoming a falling hazard. Adding perimeter concrete shearwalls would eliminate the need of adding the bracing elements since the new concrete walls would provide anchorage for the cladding.

3. Parapets

Due to their precarious location, dimensions, and elevation, the parapets around the perimeter of the structure are at particular risk. Parapets in general can experience larger horizontal accelerations due to seismic activity than other portions of the structure. To preserve the parapets and prevent serious falling hazards, a parapet bracing system is recommended at the roof level. Detail 9/S-8 in the Appendix shows a recommended method of anchoring the parapets against tipping failure.

4. Exterior Stacked Columns

An additional feature of the building that is particularly vulnerable to seismic damage or collapse are the granite columns on the south, east and west exteriors of the building. These columns are assembled from stacked sections of granite that are doweled together by at single 1" diameter steel rod. These column assemblies support gravity loads from the roof and attic levels of the structure and are not likely to perform in an adequate manner under the characterized seismic hazard. Analysis indicates that these columns begin to become unstable at a lateral acceleration of approximately 0.2g. Of the options being considered for the recommended reinforcement of these columns, the most cost effective appears to be the addition of stainless steel diagonal dowels in epoxy which connect each segment of column to the next. As a minimum all of the stacked granite columns at points of egress should be reinforced in this manner. It is recommended that all of the stacked granite columns on the exterior be retrofitted in this way to reduce the possibility of severe structural damage. This is another portion of the building that could greatly benefit from a base isolation system. By lowering the horizontal accelerations that the building could experience, the likelihood of an instability of the stacked granite columns decreases.

An alternative method that could be used to stabilized the stacked granite columns is to tie the stacked assembly together using a center coring method. To do this, the center of the column would be cored using standard coring equipment of specific diameter. The core would then be replaced with either a reinforcing rod or a reinforcing, post-tensioned tendon anchored in grout. This method of reinforcing the stacked columns would be visually less intrusive and would have virtually no affect on the current aesthetics of the exterior of the building. Due to the configuration of the space between the attic and roof and also the configuration of the parapets, this coring method may not be a feasible solution. It is recommended that this be explored in the next phase of the project to determine the feasibility of this option.

A third option would be to tie the stacked columns to the structure to effectively reduce their unbraced lengths and thus their likelihood of buckling. Though this method of bracing the columns could be visually displeasing, it would be an effective method of bracing the columns and minimizing the likelihood of buckling.

Details 5/S-8 and 8/S-6 (see Appendix) show the recommended options of stabilizing the stacked columns against buckling failure due to the anticipated seismic event.

5. Skylight Assemblies

The skylight assemblies above the east and west rotunda spaces and also above the legislative, senate, and supreme court chambers consist of both an upper and a lower system of panes. Although the skylights appears to behave well under normal service loads, there is serious concern regarding the potential damage and falling debris that could occur as a result of seismic activity. Since there are large roof and attic openings at this level of the structure that could deform in an oblique manner, the skylight infills could rack, which means that they would also deform in an oblique manner. If this were to happen, the bond between the glass panes and steel bars could break and large pieces of glass or entire panes could fall and injure occupants within areas of egress below. To mitigate the possible loss of life due to the seismic behavior of the skylights several options are available. The first option is to replace the skylights with a new skylight system that is capable of large, racking type deformations without failing or a system with tempered glass. The second option is to add a translucent laminate to the skylight system. Such laminates are used in blast retrofitting of existing structures. These options are recommended if the fixed base shearwall option of retrofit is chosen for the building. A third option is to rely on the proposed seismic base isolation system to reduce the level of lateral accelerations to the point at which the skylight system experiences minimal damage. For this option, little (if any) retrofit measures would be required for the skylight system. Since the behavior of the existing skylight system is difficult to ascertain, further investigation and analysis of the skylight system and the possible alternatives are recommended in the next phases of the project.

6. Interior Components

Since many of the interior architectural and structural components of the facility are considered to be of particular value and importance, measures should be taken to ensure appropriate anchorage of these elements to structural framework. Although base isolation could lessen the amount of load that the nonstructural elements and appendages experience, many interior components could become damaged beyond a state of feasible repair if not adequately anchored to sound structure. These items include (among other things); suspended ceilings, decorative columns, stairs, balustrades, statuary, marble cladding and marble detailing at the rotunda. It is recommended that these items be investigated in the next phases of the project to determine the amount and nature of bracing and anchoring that will be needed to meet the rehabilitation objective.

7. Stairs

The monumental stairs at the east and west ends of the rotunda are supported by reinforced concrete slabs and columns that connect to the structure from level to level. Since the stair supports are very rigid they tend to provide considerable stiffness from floor to floor and therefore could be subject to considerable seismic loading. Since these stairs are in paths of primary egress, measures should be taken to reduce the amount of seismic force that the stair supports experience, thus enabling them to remain intact. The suggested method of accomplishing this is to detach the stair assembly from being rigidly connected at the lower floor level. This will enable the stair assembly to 'ride' with the upper floor and float on the lower floor without experiencing significant seismic load.

8. Interior Marble Columns

An additional distinguishing feature of the Capitol Building requiring further study and attention in terms of seismic performance are the monolithic marble columns that line the perimeter of the rotunda and the east and west atriums. Since these columns are monolithic, they are, in and of themselves very rigid and can safely withstand considerable axial and bending loads. However, the plans indicate connections of questionable adequacy for the anchorage of these columns to the structure. Available plans indicate that the columns are connected to the structure of the second floor by a single dowel 1" in diameter that engages the web of a wide-flange shaped beam which is grouted into a pocket in the column. Although the dowel and the other steel elements comprising this connection may have adequate strength to support marble column under the characterized seismic hazard, the grout and pocket which is embedding these steel elements is of questionable capacity. In addition, the available drawings do not clearly indicate the attachment of the structure to the top of the marble columns. Based on similar conditions elsewhere in the building, the third floor most likely rests upon the top of the columns without a secure or positive anchoring mechanism. Under the characterized seismic hazard, these columns could remain intact, but may become detached from the structure they are supporting thus becoming a falling hazard in areas of primary egress. This is yet another of the buildings many features that could benefit from the recommended base isolation system. If lateral accelerations could be sufficiently reduced, retrofitting and anchoring of the monolithic marble columns may not be needed. Future and possible destructive investigation of this condition is recommended to ascertain the exact nature and capacity of the structure and elements that laterally support the marble columns.

9. Additional Life Safety Risks

In addition to the items mentioned above, nonstructural elements that require consideration as potential life safety risks include:

- Unreinforced masonry partitions 8 feet tall or more and hollow clay tile partitions.
- Nonstructural elements that may fail and impede egress.
- Hazardous materials.

These items have been identified and listed under NISTIR 5382 as nonstructural items that shall be considered as life safety risks. Further investigation is required on the Utah State Capitol to determine if elements such as these are present and to explore possible methods of mitigation. It is recommended that each of these items be carefully examined and appropriately addressed during the next phases of the project.